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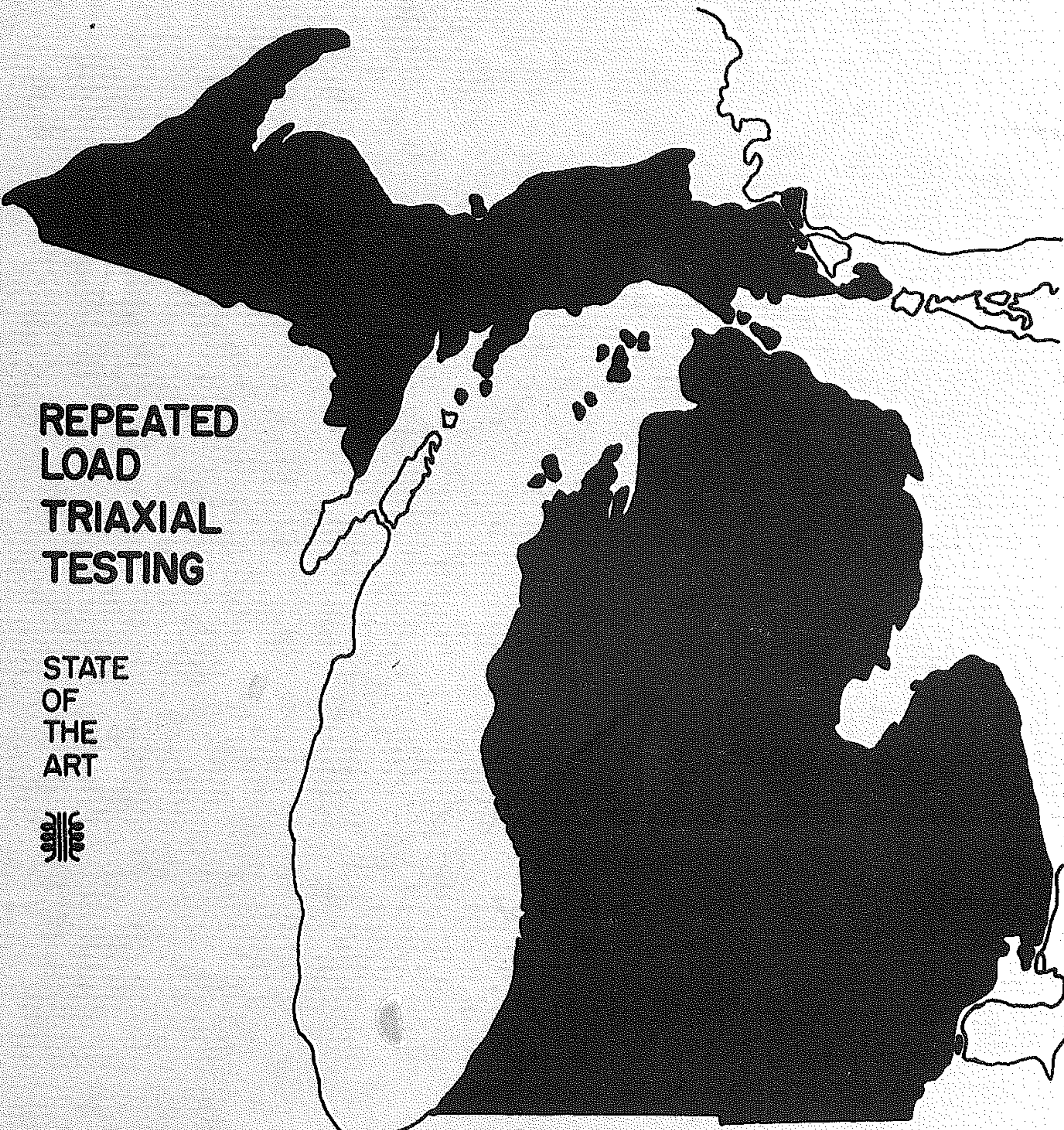

REPEATED LOAD TRIAXIAL TESTING

STATE OF THE ART

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by

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Department of Civil Engineering

STATE OF THE ART REPORT OF RESEARCH CONDUCTED UNDER RESEARCH GRANT 75-1679

RESILIENT MODULUS AND DAMPING RATIO IN CORRELATION TO SOIL SUPPORT VALUE FOR MICHIGAN ROADBED SOILS

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ABSTRACT

In the past, design and/or rehabilitation of flexible highway pavements were based on a rule of thumb procedure and the accumulated experience of the highway engineer, with the result that severe break-up was a common occurence. Thus, the need for new design methods and improved material characterization techniques were frequently stated. Recently researchers recognized the fact that the action of traffic on highway pavement is a transient one. Consequently, they established a dynamic, repeated load testing technique as a tool for the characterization of highway materials.

In this paper, an attempt is made to review some of the available literature dealing with dynamic testing and the many variables affecting the test output. The subject is presented in seven chapters as outlined in Chapter 1. Appendix A is devoted entirely to test procedure as published in Special Report 162, Transportation Research Board, National Research Council, National Academy of Sciences, Washington, D.C., 1975.

CHAPTER 1

INTRODUCTION

The complexity and variability of pavement-subgrade materials and their interactive mechanism make the design and/or rehabilitation of an existing pavement a major problem. Present design methods are empirical and quasi-rational; they are based on correlation with in-service performance (1, 2, 3 and 4)*. These design procedures consider only a few material descriptors, and cause great difficulties in extrapolating and correlating pavement performance under different loading and environmental conditions. Thus, the need to develop an approach to material characterization, which recognizes the complexity and variability, not only of the individual pavement components and their interaction, but also the conditions that exist throughout the life cycle of the pavement system, has been frequently stated (5,6).

Further, present design of flexible highway pavements has been, for many years, based on the accumulated experience of the highway engineer (3), with the major design consideration being control of permanent deflection. While permanent deflection is one way in which the pavement can fail, there are five other modes of distress which must be considered in the design process. These modes of distress include:

1) fatigue, which occurs in the layers of the flexible pavement structure, is caused by the repeated bending of the layers due to traffic traveling over the pavement surface,
2) rutting, which is caused by cumulative plastic and shear deformations in the subgrade and/or base materials as a result of load repetitions,
3) excessive deflection in the base materials due to compaction by vehicular loads,
4) temporary excessive rebound in the subgrade and base materials (7), and
5) lack of stability in the wearing course (and resistance to fatique failure.)

These failure modes are manifestated by an uneven surface and a pavement can be considered to have failed functionally when deformation of its components are sufficiently large to cause an unacceptable and uneven riding surface, or to cause cracking of the surface material (2).

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* Figures in brackets indicate references in the Bibliography

In recent years, those in the field of highway design have called for a new improved design method, which will deal with all modes of distress which can deteriorate flexible pavements. This new method of design has often been referred to as the rational design method. It is a more realistic approach to design, based on the mechanical properties of the roadbed and subgrade materials. Efforts to perfect this method of design have been focused in two areas. The first of these is proper characterization of the materials. The second, which is based on the first, is technique whereby deflections of the pavement may be predicted.

The characterization of paving materials and subgrades is a complex task. Formerly, these materials were characterized by their static behavior, i.e., the loads applied to materials being tested were static, even though their magnitudes may have been subject to change during the test. The California Bearing Ratio (CBR), the Hveem stabilometer, and the static triaxial test are representative of this type of test. However, the application of stress to pavement materials by moving wheel loads is a transient one. A more realistic test procedure to characterize these materials should be one in which the loads applied to specimens are also transient. The repeated load triaxial test is one such test. Samples of soil or paving material are placed in the cell and subjected to confining and axial stresses, just as in the static triaxial test. The difference, however, is that the application of stresses to the sample in the cell is cycled or repeated. The repeated application of axial stress does not duplicate applied stresses in the field, but more realistically represents the form of stress applied to roadbed materials by traffic. Some of the drawbacks involved with this test will be discussed later in this paper.

Reliable predictive techniques must be available to implement the rational design method. Though some of the problems in predicting pavement deflections stem from the difficulties in properly characterizing the roadbed materials, still more problems are encountered by trying to select an appropriate theory to make this prediction. Recent investigations employing the transfer function theory seem to have considerable success. Baladi (5), using the transfer function theory and the Kelvin-mass-spring-dashpot model, predicted pavement deflections to within five percent (5%), 0.0005 inch, of the measured deflections for nine different flexible highway and runway pavements.

A new rational design method is sought which deals with the mechanics of the pavement materials on a more elemental level. Researchers believe a more efficient design may be realized through the association of calculated stresses with the mode of failure presented. The development of a new design method has been impeded primarily by two aspects of analysis: accurate material characterization and reliable pavement deflection prediction. The behavior of soils under repeated loading is much different from that under static loading. Accurate material characterization requires that investigators try to simulate field states of stress in testing, or to use full-scale field tests in a rapid nondestructive manner (3, 8, 9, 10, 11, 12, 13, 14, and 15).

This paper will deal with efforts by investigators to characterize roadbed materials through the use of repeated load triaxial testing. The subject will be presented in the following order:

1. Chapter 2 Review of Literature
2. Chapter 3 The Repeated Load Triaxial Test
3. Chapter 4 Factors Affecting the Resilient Modulus
4. Chapter 5 The Stiffness Modulus of Asphalt Treated Mixes
5. Chapter 6 Sample Preparation and Test Procedure
6. Chapter 7 Equipment
7. Appendix A Test Procedures for Characterizing Dynamic Stress-Strain Properties of Pavement Materials (Special Report 162, Transportation Research Board, National Research Council, National Academy of Sciences, Washington, D. C., 1975).

CHAPTER 2

REVIEW OF LITERATURE

In the early stages of development, design and/or rehabilitation of a pavement system consisted of rule-of-thumb procedures based on judgment and past experience. In the 1920's, the U.S. Bureau of Public Roads* developed a soil classification system based upon the observed field performance of soils under highway pavements (16). This system, in conjunction with the accumulated data, helped the highway engineer to correlate performance with subgrade types.

Beginning in the late 1940's engineers were faced with the need to predict the performance of pavement systems subjected to greater wheel loads and frequencies than they had ever before experienced (3,4,17). Thus, a quasi-rational design procedure was introduced in the early 1950's (18); however, severe breakup is still a common phenomenon on some flexible highways and runways (18,19).

An important problem which the highway engineer faces today is that of providing remedial measures to upgrade existing pavements to meet today's traffic loadings and frequencies. This need has led many investigators to agree that a closer look at the materials comprising the pavement structure is a must. Researchers concerned with fatigue failures recognized the need for a testing method which would simulate the action of traffic. This was pointed out by Professor A. Cassagrande who wrote (20):

> "Irrespective of the theoretical method of evaluation of load tests, there remains the important question as to what extent individual static load tests reflect the results of thousands of dynamic load repetitions under actual traffic. Tests have already indicated that various types of soils react differently, and that the results of static load tests by no means bear a simple relation to pavement behavior."

Mitry (36) noted the work of many investigators who first began testing with repeated loads. In 1947, Campen and Smith (55), McLead (56), Phillippe and Hittle (57), and Goetz (58) had all begun investigations of repeated load tests on model pavement sections, with the number of load repetitions on the order of 10. However, due to several disadvantages of the test (time consumption and cost), experimentation with repeated load testing in the conventional triaxial cell was soon recognized as a better test.

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* The Bureau of Public Roads is now known as the Federal Highway Administration. (3)

The cyclic (repeated) plate load tests could only evaluate the soil parameters under one set of conditions, namely, those that existed at the time of testing. However, critical soil conditions could be reproduced in the triaxial cell. This strengthened the practice of material property determination in the laboratory. The effects of many different parameters such as density, gradation, degree of saturation and others were soon under investigation.

The first efforts in triaxial testing were associated with the evaluation of repeated load characteristics of subgrade materials such as clays and silts. According to Mitry (36), Barber presented data in 1959, which showed that increased fines content in aggregates considerably decreased its permeability. The need for drainage time was recognized, due to the development of excess pore water pressure upon loading. Seed and Chan (6), showed that the resilient modulus of the silts increased as the time of duration of the axial load decreased.

Given the considerable amount of data available from conventional static tests, a correlation between dynamic and static test properties was sought. Seed et al (6) made a comparison between Young's Modulus as determined by the unconfined compression test and the resilient modulus. Figure (2.1) shows that the resilient modulus in all tests was higher than the tangent modulus for tests on silty clays. Ahmed and Larew (21) found just the opposite. In tests on silts and clays, they determined the strength and modulus by conventional tests. They ran repeated load tests using 6 different levels of repeated stress which were less than the determined strength. In all cases, the modulus based on the repeated load test was less than that for the static test as shown in Figure (2.2). The results also show that stiffness and peak strength were less in the repeated load case.

Repeated load tests on poorly graded sand, with a slow cyclic frequency to represent loads due to parking were performed by Trollope (59). In these tests, he found that the resilient modulus increases with increasing dry density (decreasing void ratio) and increasing rate of deformation. Hicks (29) and Mitry (36) acknowledged Biarez (60) as the first investigator to note a logarithmic relationship between the resilient modulus, M_R, and the sum of principal stresses, θ. In tests on uniform sand, a log-log plot

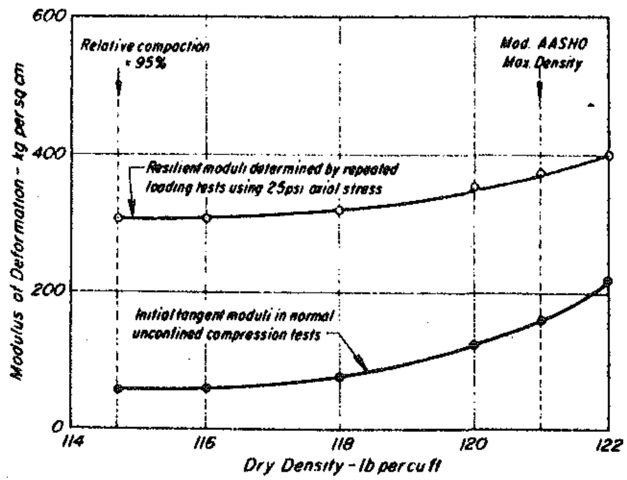


FIGURE 2.1 COMPARISON OF RESILIENT MODULUS AND TANGENT MODULI FOR SILTY CLAY (6).

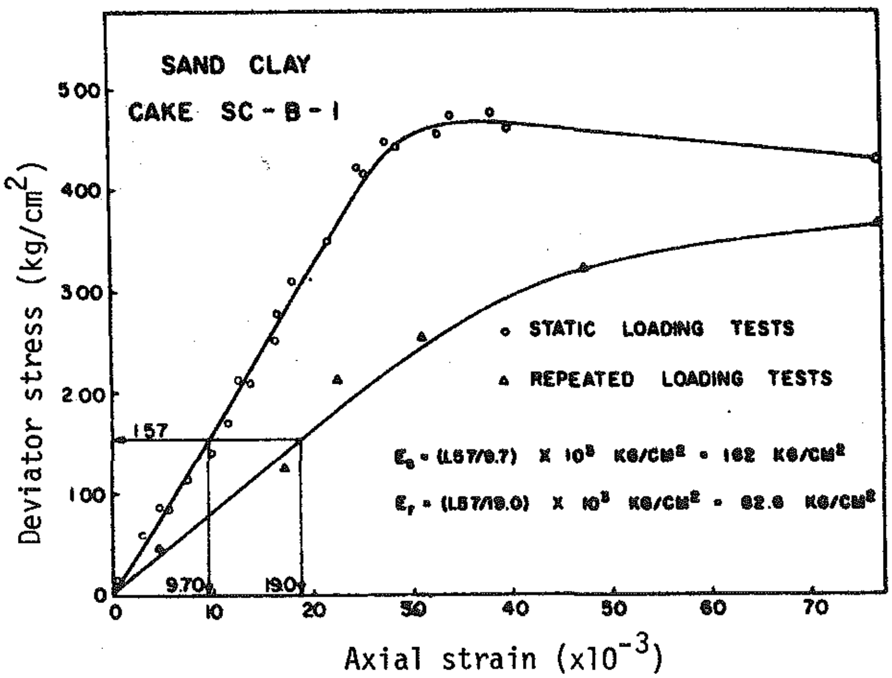


FIGURE 2.2 STRESS STRAIN CURVES (21).

of M_R vs. θ gave a straight line, which could be expressed by the equation:

$$M_R = K \cdot \theta^n \tag{2.1}$$

in which K is a constant, and n is an exponent between 0.5 and 0.6. Dunlap (61) formulated another relation between the resilient modulus and the triaxial stress level as follows:

$$M_R = K_1 + K_2 (\sigma_r + \sigma_\theta) \tag{2.2}$$

in which, K_1 = the unconfined modulus

K_2 = a constant

σ_r = the radial stress

σ_θ = the tangential stress

Mitry (36) performed many tests on untreated base course material. He confirmed the linear relationship, on a log-log scale, between resilient modulus and the confining pressure. He expressed the relationship in terms of the confining pressure as,

$$M_R = K\sigma_3^n \tag{2.3}$$

in which, σ_3 = the confining pressure

K = a constant

n = an exponent between 0.5 and 0.7

He noted the strong effect that the confining pressure has on the resilient modulus, one increasing with the other. He also found that the resilient modulus for saturated gravel under drained conditions was only slightly higher than that for dry specimens, and that the resilient modulus determined under undrained conditions was nearly the same as that of dry aggregate. Seed and others confirmed this in 1967. Coffman (62) showed that the resilient modulus increased with increasing frequency of repeated load.

Morgan (36) conducted tests on uniform sand. He reported that the behavior of freely drained saturated sands is only slightly different from that of the air-dried sands. Morgan found that the resilient modulus is dependent on the magnitude of the deviator stress and confining pressure. Also, he found that the resilient Poisson's ratio was unaffected by changes in either of the test parameters.

Haynes and Yoder (26) carried out repeated load triaxial tests on different kinds of coarse aggregate, gravel and crushed stone. They found that the resilient modulus decreased with the increasing saturation. The amount of decrease was dependent on the aggregate type; gravel being affected more than the crushed stone. The resilient modulus was found to be only slightly affected by gradation.

Tests made by the Asphalt Institute in 1967 on untreated base course materials also showed that the resilient modulus decreased with increasing saturation. Hicks (29) cites the findings of Kallas and Riley (63), which saw the decrease of K while n remained constant in equation (2.3). In tests run by Kasianchuk (64), the build-up of excess pore water pressure and a corresponding decrease in effective confining pressure, with an increasing number of load applications was reported. These tests were made on saturated sand and appear to be related to the phenomenon of liquefaction. Kasianchuk also confirmed the linear relationship between confining pressure and resilient modulus. For a comparison of these findings, see Table (2.1).

In 1962, Seed showed that the resilient modulus of clay was dependent on axial stress level. Recognizing this fact, the characterization of the subgrade layer becomes very complex. Since the load on the soil varies with horizontal and vertical position, the resilient modulus varies throughout the soil no matter how homogeneous it may be.

The emphasis of researchers seems to have changed at this point. With a great deal of testing having already been done, it was fairly clear how many parameters were affecting the resilient response of highway materials. It was now a matter of determining which test parameters and conditions were most important. Hicks (29) addressed himself to this in work on untreated base course materials at the University of California. He showed that stiffness increased (resilient modulus decreased) with increasing confining pressure, and was relatively unaffected by the deviator stress. Stiffness increased with density, decreasing fines and decreasing saturation. The magnitude of the increase in stiffness was dependent on the type of aggregate tested. The resilient Poisson's ratio increased with decreasing confining pressure and increasing deviator stress. These tests were carried out in a conventional triaxial cell, with repeated axial stress and sustained confining pressure. Axial and radial strains were measured, based on realistic stress histories, load duration and frequency, at stress levels expected in the field.

TABLE 2.1 SUMMARY OF THE COEFFICIENTS K_1, K_2, K_1', K_2', RELATING MODULUS TO CONFINING PRESSURE (σ_3) AND TO THE SUM OF THE PRINCIPAL STRESSES (θ), (27).

REFERENCE	MATERIAL	$M_R(psi) = K_1 \sigma_3^{K_2}$		$M_R(psi) = K_1' \theta^{K_2'}$		WATER CONTENT
		K_1	K_2	K_1'	K_2'	
(36)	Dry Gravel	7,000	.55	1,900	.61	-
(112)	Crushed Gravel	13,000	.50	-	-	.20
		9,000	.50	2,800	.59	.07
(64)	Aggregate Base	11,300	.39	3,830	.53	
	Aggregate Subbase	6,310	.43	2,900	.47	
(63)	Aggregate Base and Subbase	10,618	.45			.024
		10,144	.47			.043
		10,019	.47			.063
		8,687	.50			.082
(113)	Aggregate Base			5,400	.50	.027
				2,100	.50	.063

As a result of this work, Hicks and Monismith (28), reported that the effects of stress level on the resilient modulus are greater, than those for other material parameters such as density, gradation, and saturation, which have a lesser importance. Hicks and Monismith also noted a relationship between resilient Poisson's ratio and the principal stress ratio of the form,

$$\nu = A_0 + A_1 (\sigma_1/\sigma_3) + A_2 (\sigma_1/\sigma_3)^2 + A_3 (\sigma_1/\sigma_3)^3 \qquad (2.4)$$

in which A_0, A_1, etc. are regression constants from a least squares curve fitting, and (σ_1/σ_3) is the principal stress ratio.

Barksdale and Hicks (23) suggested a relationship between the measured plastic strain in a repeated load test and rutting of the surface of a flexible pavement. They defined the rut index as "the sum of the average plastic strains occurring in the top and bottom half of the base multiplied by a constant 10,000 so as to give a whole number." They stated that the rut index can be evaluated from the results of two repeated load triaxial tests performed at a confining pressure of 10 psi and deviator stresses of 35 and 60 psi. However, they acknowledged that a more general approach than the rut index must be used to study rutting in pavement structures having different geometries and varying base course materials. To this end, it was apparent that a proper material characterization was still not in hand. Thus, investigators started looking for other methods to determine the stress-strain relationship for asphalt mixes; among these methods are the stiffness modulus, the complex and/or dynamic modulus, and the dynamic stiffness modulus.

Nijboer (65) related asphalt mix stiffness to the ratio of Marshall stability and Marshall flow value. Terrel et al (47) cited the work of other investigators in the correlation of the ultimate tensile strength with the resilient modulus. In the estimation of mix stiffness, Heukelom and Klomp (69) extended the earlier work of Van der Poel (70,71). They presented a semi-empirical equation whereby the volume concentration of aggregates is determined. It should be noted that these equations also make use of the nomographs presented by Van der Poel.

The concept of stiffness modulus was first presented by Deacon (68), based on the results of repeated load beam flexure tests; however, it was Terrel who presented the most noteworthy work in resilient and complex modulus determinations in the triaxial cell in 1967 and again in 1972.

The most significant of his findings is the linear stress-strain behavior of asphalt treated mixes in the range of stresses and temperatures expected in the field, in contrast to the non-linear behavior of untreated soils. Terrel and Awad (47) stressed the continuation of research to develop and refine more realistic test methods, pointing out the failure of conventional testing in newer theoretical techniques with adequate material parameters. Recently, investigators have recognized that pavement deflection is one such technique. Thus, the search was begun for a method whereby an accurate pavement deflection can be made.

In the search for the theoretical basis upon which to predict pavement deflections, Pell and Brown (41) gave further support to linear elastic theory, noting it as the most promising. First, this theory could be modified for use with non-linear material properties, such as exhibited by cohesive and granular soils, through an iterative process. Second, the thickness of the flexible pavement usually insured linear behavior. Interaction between layers, which is a function of the layer thickness and material composition is as important as the behavior of the materials comprising the pavement structure. Furthermore, the use of a linear elastic theory may introduce nonpredictible error. To this end, layered elastic theory was introduced. Seed et al (44) had limited success in prediction of pavement deflection based on layered elastic theory and laboratory determined properties.

In 1970, Harr introduced the transfer function theory as a method whereby pavement parameters could be determined. Ali (72) applied transfer function theory to study flexible pavements under laboratory controlled conditions. He reported: that "Temperature, surface course thickness, and spatial location have their respective influence on the transfer function." Boyer and Harr (73) extended transfer function theory to an in-service pavement system. They used installed linear variable differential transformers (LVDT) gages in the pavement and conducted field tests at Kirtlind Air Force Base, New Mexico. They concluded that the characteristics of flexible pavements could be represented by a "time dependent transfer function" (TDT). Baladi (5) and Ng-A-Qui (74) successfully predicted pavement deflection of various highway and runway sections using the transfer function theory. Also, Baladi succeeded in determining pavement parameters that are needed in the design and/or rehabilitation of flexible pavement structures.

In summary, the effort to develop a new rational design of flexible pavements or to modify existing design methods has been concentrated in two areas: 1) characterization of roadbed and surface materials, and 2) development of a technique whereby an accurate prediction of pavement deflection can be achieved. Most researchers agree that the theoretical techniques for prediction of pavement deflections are far more advanced than our ability to characterize the paving materials. Indeed, the problem of predicting pavement response is primarily related to the lack of adequate material parameters. Due to the complexity and variability of highway materials, and the limitation of our testing ability, much of the work in this area has been of little help in changing and improving methods of design. However, research has increased our knowledge about the problem; the different modes of distress and the mechanics behind them have been identified; the testing procedure has been improved such that field conditions are now being accurately simulated in the laboratory. Further, full-scale field testing is a must in order to modify the laboratory test procedures and consequently to check its results.

CHAPTER 3

THE REPEATED LOAD TRIAXIAL TEST

1. INTRODUCTION

The repeated load triaxial test has been singled out for extensive use by researchers for material characterization. There are several reasons for this:

1) it is considered the best test technique due to the ease with which investigators can control various parameters,

2) low cost, and

3) the ability to give relatively accurate estimations of material properties for pavement deflection analyses.

However, as we shall see, the repeated load triaxial test falls short of truly representing the real in-situ state of stress for the soil element. Before examining the inconsistencies of the state of stress, the repeated load triaxial test itself will be considered.

2. THE TEST

Triaxial testing is concerned with the determination of the stress-strain behavior of soils. Cylindrical soil samples are carefully prepared, most often to represent the in-situ condition of the soil, and these samples are placed in a test chamber or cell. In the cell, the soil sample is subjected to a lateral or radial confining stress, and an axial stress applied by a piston to the end of the sample. In conventional triaxial testing, the sample is subjected to an axial stress which is maintained and steadily increased. Stresses and strains are monitored throughout the test. Repeated load testing is much different in several ways. The soil sample in the cell is still subjected to a confining stress by pressurizing the air or cell fluid in the chamber, but the cell pressure may be pulsated as is the axial stress. The axial stress, which in conventional testing is maintained, is continuously reapplied to the specimen. This repeated loading of the sample is meant to represent the stress pulse felt by a soil element due to moving wheel loads on pavements. If the test confining pressure is pulsed, it is typically pulsed in sequence with the axial load. Thus, many more test parameters require consideration in repeated load testing. Not only must values of the axial and confining stresses be specified, but the conditions of loading such as load frequency, duration, and number of loadings must also be considered. As might be expected, each of these test parameters has an effect on the response of the material. These effects will be discussed in the next chapter.

3. STATE OF STRESS

To illustrate the manner in which the triaxial test fails to truly represent the state of stress of a soil element in the pavement structure, consider the stresses induced in the element due to the passage of a moving wheel load. Figure (3.1) shows that as the wheel moves along the surface of the pavement, the orientation of the principal stresses which are applied to an element in the pavement structure rotates. At an instant when the load is directly above the element, the principal stresses are oriented horizontally and vertically. Except for this very instant, the major principal stress applied to the in-situ element is at all times greater than the vertical stress. The triaxial test employs application of principal stresses in one orientation only, that of horizontal and vertical, with no possibility for reorientation. Due to the fact that the principal stress applied by the wheel load rotates as the wheel moves, a shear stress, τ, exists on the vertical and horizontal planes of the in-situ element. Figure (3.2) illustrates the normal and shear stresses exerted on the in-situ element. In the figure, the shear stress is zero when the normal stresses are maximum, corresponding to the wheel load located directly above a soil element. The triaxial test can only represent this condition, since it is incapable of applying shear stresses directly to the sample. Deformations of the sample are measured only in the directions of the applied normal stresses, giving an inaccurate and overestimated measure of permanent deformation. However, in relation to the in-situ element, Pell and Brown (41) stated that if the soil is considered to be isotropic, the measured deflection can be considered satisfactory.

In representing the state of stress of an element under the surface of a pavement, the triaxial test falls short of a true representation in two ways:

1) the principal stresses on an element in the field rotate, whereas the repeated load triaxial test applies them in one orientation only, and
2) because of the rotation of the principal stresses, shear stresses

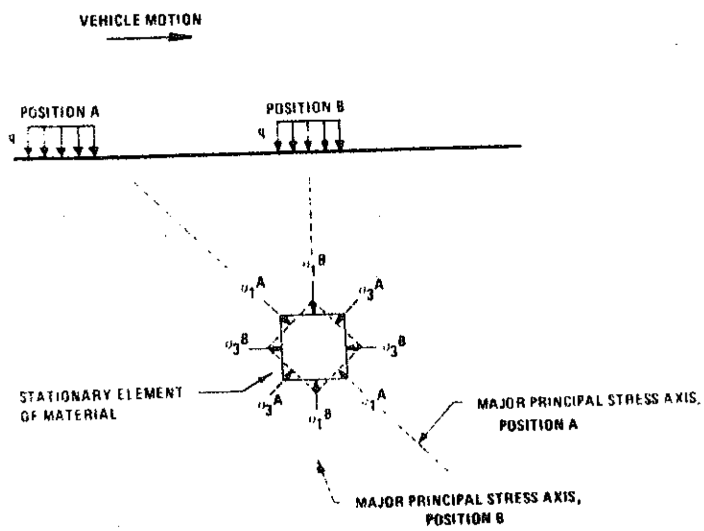


FIGURE 3.1 ROTATION OF PRINCIPAL STRESS AXES OF AN ELEMENT AS A VEHICLE MOVES ALONG THE SURFACE OF A PAVEMENT, (24).

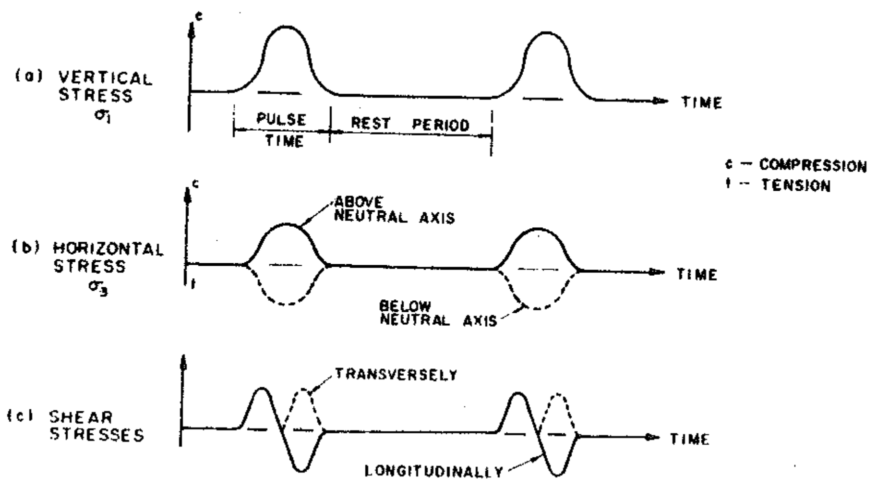


FIGURE 3.2 STRESS VARIATIONS IN A TYPICAL ASPHALT CONCRETE ELEMENT DUE TO A MOVING LOAD, (40).

occur on the horizontal and vertical planes of the element

These shear stresses cannot be applied in triaxial testing. No one is certain as to the importance of these drawbacks in material characterization, however, it does not appear likely that the repeated load triaxial test will be replaced in the near future. Morgan (39) suggested the use of direct shear testing to supplement triaxial testing characterization, but it is not clear how this would be achieved. Baladi (5) has suggested the use of full-scale field tests in conjunction with repeated load triaxial tests so as to determine the importance of these shortcomings of the triaxial testing. Before looking more closely at the various triaxial test parameters, the definition of some terms may be warranted.

4. RESILIENT PROPERTIES

In the determination of the most important resilient properties, the response of the test specimen is carefully monitored. Axial and radial strains and deformations characterize this response. When the sample is loaded axially it deforms a certain amount, and upon unloading, a portion of this total deformation is recovered. Thus, the total deformation is comprised of an elastic or recoverable deformation and a permanent deformation. These deformations lead to the corresponding total, resilient, and permanent strains defined by them. The two most important resilient properties, the modulus of resilient deformation, M_R, and resilient Poisson's ratio, ν_r, are defined by these strains and the values of the applied stresses.

The resilient modulus M_R is defined as follows:

$$M_R = \frac{\sigma_1 - \sigma_3}{\varepsilon_a} = \frac{\sigma_d}{\varepsilon_a} \qquad (3.1)$$

in which, σ_d = the deviator stress which is the difference between the axial stress, σ_1, and the radial stress σ_3

ε_a = the resilient or recoverable axial strain

This definition applys to a linear elastic, isotropic material under uniaxial stress. It is valid for cohesive and cohesionless soils and has also been adopted to characterize asphalt treated materials. Figure (3.3) shows typical recordings of stress and strain taken for a repeated load triaxial test, and the resilient modulus determined based on them.

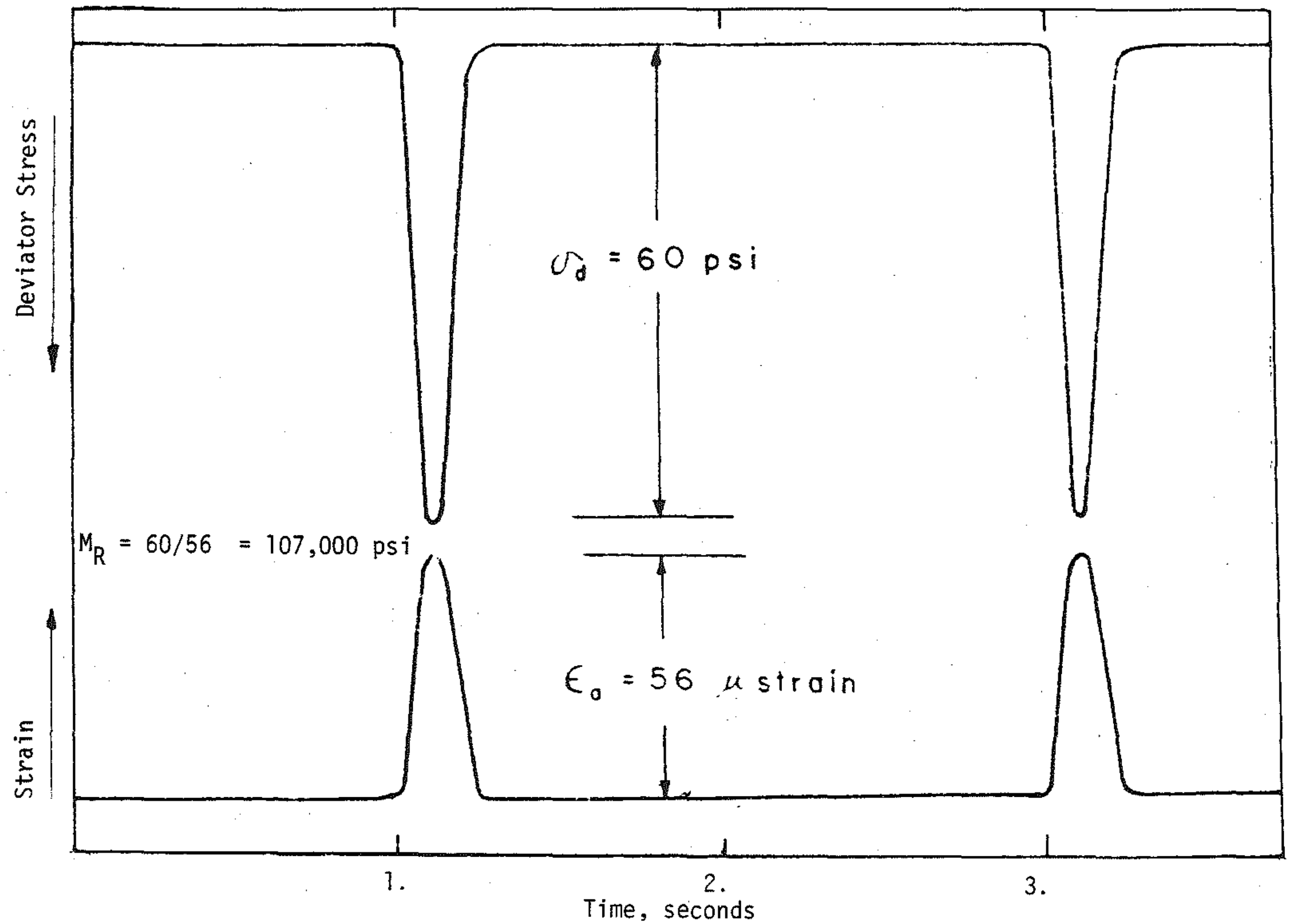


FIGURE 3.3 TYPICAL LOAD-DEFORMATION RECORDING TRACE FOR σ_3=20 psi AND FREQUENCY OF 30 cpm, (31).

The resilient response and M_R for asphalt treated materials are dependent upon many factors. Among these are temperature, mix properties, stress level, load duration and frequency. It is also known that the value of the resilient deformation measured is dependent upon load duration. Under short stress durations and low temperatures, the asphalt treated materials behave almost elastically. However, saturated mixes at high temperatures exhibit little or no resilient response, which leads to an excessively high and misleading value of M_R. Terrel and Awad (48) recognized that M_R was not enough to completely characterize the stiffness or quality of a mix. They introduced the modulus of total deformation, M_T, defined as follows:

$$M_T = \frac{\sigma_d}{\varepsilon_T} \tag{3.2}$$

in which, σ_d = the deviator stress

ε_T = the total strain

They observed that when total strains were used instead of resilient strains, the material properties computed were more consistent. Figure (3.4) shows a typical recording of stress for an asphalt treated material.

The resilient Poisson's ratio, ν_r, for isotropic linear elastic materials under uniaxial stress is defined as follows:

$$\nu_r = \frac{\varepsilon_r}{\varepsilon_a} \tag{3.3}$$

in which, ε_r = the recoverable radial strain

ε_a = the recoverable axial strain

This resilient Poisson's ratio and modulus are defined under the condition of a constant confining pressure. The most recent investigations have been made utilizing a variable confining pressure which pulses with the axial load, and more accurately simulates the stress conditions in the field.

With a variable confining pressure, determination of the resilient modulus as previously defined would ignore the effects of the pulsed radial stress. This change in confining pressure would have an effect on the recoverable axial strain, leading to an overestimate of ν_r based on the constant confining pressure. Allen and Thompson (22) suggested the use of three

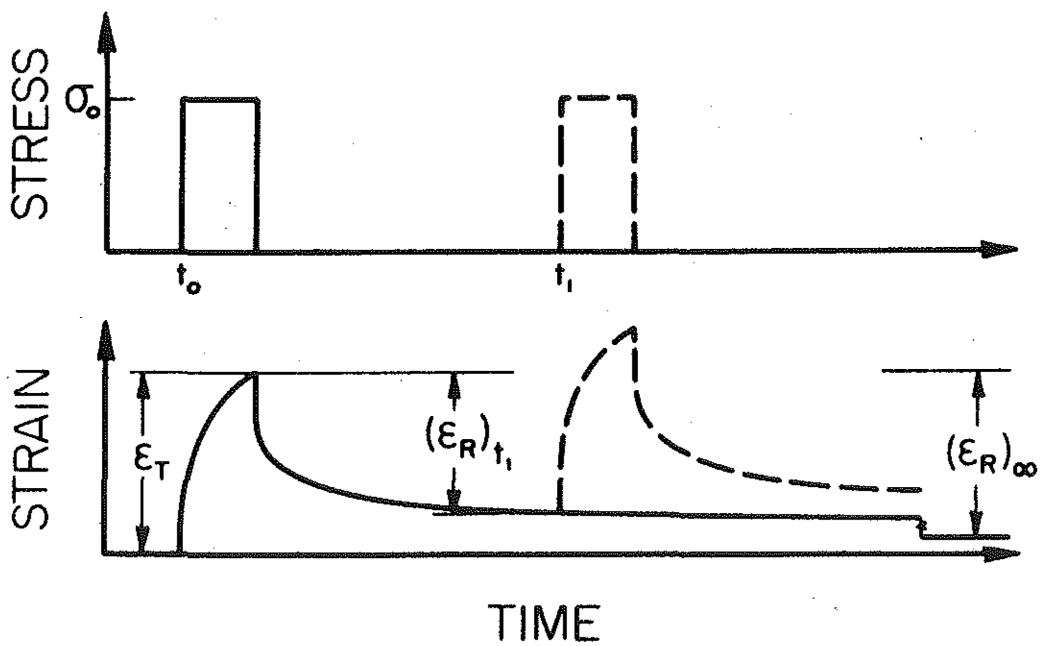


FIGURE 3.4 RESILIENT AND TOTAL STRAINS UNDER REPEATED STRESS, (47).

dimensional stress-strain relationships:

$$\varepsilon_a = \frac{1}{M_R} (\sigma_a - 2\nu_r \sigma_r) \tag{3.4}$$

$$\varepsilon_r = \frac{1}{M_R} [\sigma_a - \nu_r(\sigma_a + \sigma_r)] \tag{3.5}$$

in which, ε_a = the recoverable axial strain

ε_r = the recoverable radial strain

ν_r = the resilient Poisson's ratio

σ_a = the axial stress

σ_r = the radial stress

Terrel et al (48) presented the following equations for strain based on linear elastic behavior

$$\varepsilon_r = B_1 \sigma_r + B_2 \sigma_a \tag{3.6}$$

$$\varepsilon_a = B_3 \sigma_r + B_4 \sigma_a \tag{3.7}$$

in which, B_1 through B_4 are constants determined from linear fitting of the experimental data. If ε_r and ε_a are resilient strains, then B_1 through B_4 can be used to determine M_R and ν_r as follows:

$$M_r \approx \frac{2}{B_1 + B_4} + \frac{2}{3} \frac{(B_2 + B_3)}{\left[(B_1 + B_4) - \frac{(B_2 + B_3)}{3} \right] (B_1 + B_4)} \tag{3.8}$$

$$\nu_r \approx -\frac{2}{3} \frac{(B_2 + B_3)}{(B_1 + B_4) - \frac{(B_2 + B_3)}{3}} \tag{3.9}$$

Again, these relationships are based on linear elastic behavior and, therefore, may only be applied to materials exhibiting this behavior.

5. TEST PARAMETERS

Researchers seek to predict the in-service behavior of the pavement structure based on the results of laboratory tests on its components. It follows that testing in the lab must be performed in such a manner as to simulate the actual field loading and soil conditions. To simulate field conditions, realistic values for the various test parameters must be chosen. The ranges of stress, temperature, number and duration time of loads, etc., are chosen so that they fall within service conditions. These parameters will be discussed in the following subsections.

5.1 Load Repetitions

The choice of this parameter for testing is reasonably straight forward. One would expect to subject a test specimen to a number of loads of the same order as that which we would expect in the field. During its lifetime, a typical highway pavement can be subjected to anywhere between 100,000 to 1,000,000 or more 18 kip single axle loads. Indeed, most repeated load triaxial testing is carried out with the number of load applications in the range of 10,000 to 100,000 repetitions. As will be pointed out later, after a certain number of applications, the response of the specimen does not change appreciably. This fact, in conjunction with the excessive time required to apply a realistic number of loads is the reason that investigators use a smaller number of loadings.

5.2. Deviator Stress

It will become evident that changes in the stress level to which test specimens are subjected will affect the resilient properties to a greater extent than changes in any other test condition. The resilient modulus may vary as much as several hundred percent in the range of stresses encountered in the pavement structure. To determine the value of axial load to apply to a specimen, several things must be considered. Of primary importance is load intensity in the field. It is obvious that the intensity of the vertical load applied diminishes with depth, but the level of stress to which an element is subjected is also affected by the geometry of the pavement structure. Computer programs aid researchers in determining the magnitude of load to expect and use for testing for a given material at a given depth in the roadbed. Figure (3.5) shows the variation of stress with time for different depths in the base course for a given vehicle speed and tire pressure. Published data such as this aids investigators in choosing the appropriate axial stress level.

5.3 Load Wave Form

As the axially loading piston moves downward to make contact with the test specimen, the change of applied load with time must be con-

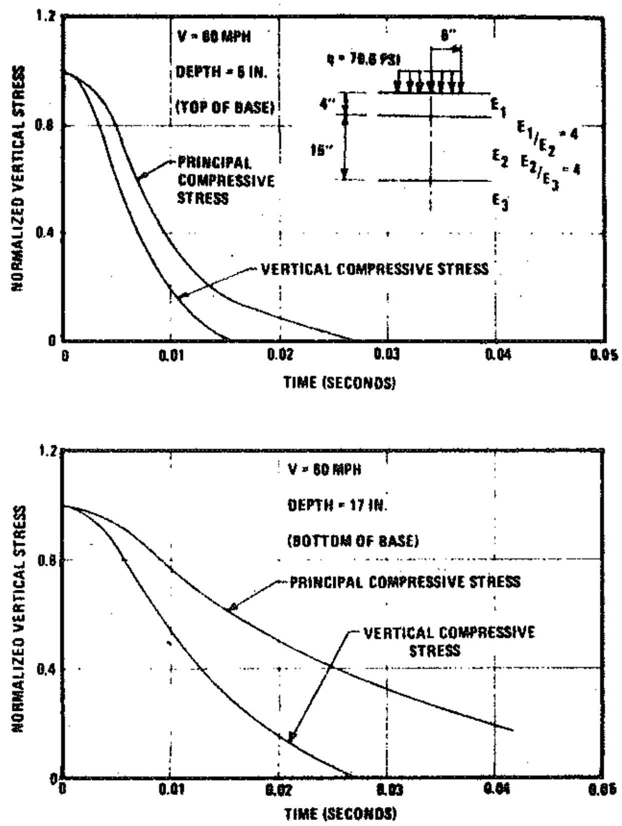


FIGURE 3.5 COMPARISON OF VERTICAL AND PRINCIPAL COMPRESSIVE STRESS PULSES FOR TWO DEPTHS, (4 inch surface and 15 inch base), (24).

sidered. This is typically referred to as the wave form. Figure (3.6) shows some of the wave forms used in past investigations. The most commonly used stress-pulse for many years was the square wave. It was used because analysis of the data was simplified and it was a wave form which was easily achieved with pneumatic loading test systems used in many laboratory studies.

Barksdale (24) found that the form of the stress pulse changed with depth for in-service loadings. He found that the vertical stress pulse varies from a near sinusoidal one near the top of the pavement structure to a more nearly triangular pulse in the lower portions of the base course. Figure (3.7) shows the variation of stress versus time for different depths in the pavement structure. Examination of the figure indicates that a triangular or sinusoidal wave form may be considered as a good approximation. Terrel and Awad (47) were in agreement with these findings. They noted the replacement of sinusoidal waves with other wave forms, by researchers, for ease in analyses.

Allen and Thompson (22) also showed the dependence of stress wave form upon depth. In agreement with Barksdale and Terrel, they described the vertical stress pulse as generally sinusoidal, with a sharper peak near the surface, and a flatter top in deeper portions of the base course. Conversely, they claimed that the radial stress pulse was more or less a flat-topped sinusoidal shape, which became more sharply peaked with depth. For all of their testing, a half-sinusoidal wave form was used since this shape is the most general for all of the different stress distributions, and because it can be produced with standard laboratory function generators.

In work performed by Terrel et al (48) on the effects of different wave forms on specimen response, it was found that no significant difference was observed in the total and resilient strains using either the sinusoidal or triangular wave shapes. Also, an equivalent square pulse can be used if the same stress is applied for 33% of the duration of the equivalent sinusoidal pulse, or if 66% of the sinusoidal stress is applied for the same duration as the sinusoidal pulse. For simplified data analysis, and other reasons, they concluded that a square vertical wave form is a

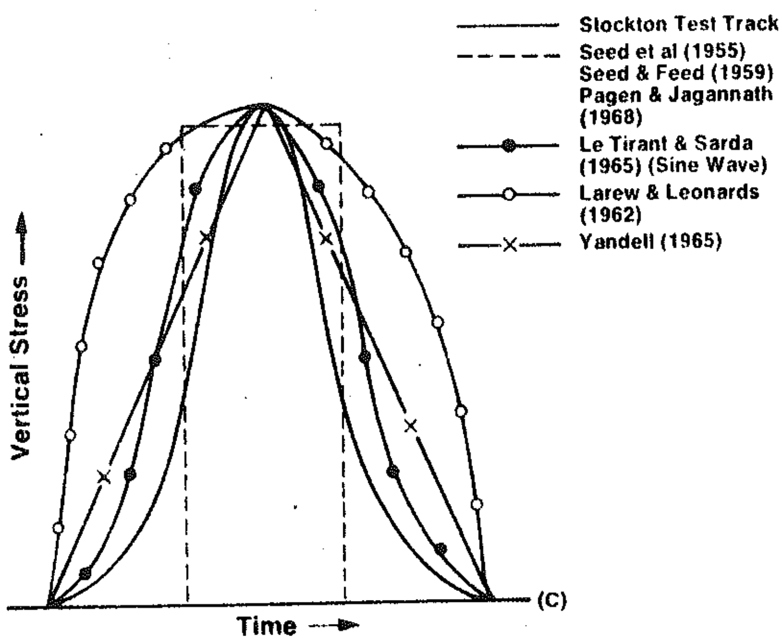


FIGURE 3.6 VERTICAL STRESS FUNCTIONS USED BY DIFFERENT INVESTIGATORS, (45).

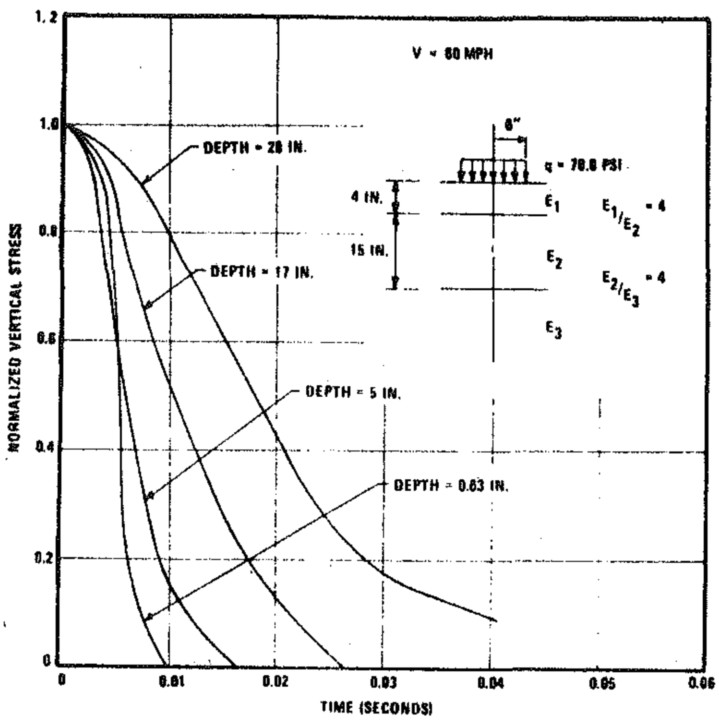


FIGURE 3.7 VARIATION OF CALCULATED VERTICAL COMPRESSIVE STRESS PULSE SHAPE WITH DEPTH, (4 inch surface and 15 inch base), (24).

reasonable approximation of actual conditions. Considering the deficiencies and imperfections of laboratory testing and predictive techniques for deformation, the precise wave was of little importance. They suggested the use of the sinusoidal wave pulse for all testing, which still appears to be the case today.

5.4. Load Frequency and Duration

Under actual in-service conditions, the stress pulse applied by a moving wheel lasts about 0.01 to 0.1 of a second. This duration time is primarily dependent upon the speed of the vehicle and the position of the element under consideration within the pavement structure. The vehicle speed is inversely related with the load duration. As vehicle velocity increases, the duration of loading decreases linearly, and as the velocity decreases, the load duration linearly increases. It is also known that the load duration time increases with depth. For a flexible pavement with 4 inches of surfacing and 15 inches of base course, Barksdale (24) found that the time of load duration increases by a factor of about 2.7 from surface to subgrade.

As was pointed out earlier, the principal stresses applied to the in-situ soil element are always greater than the vertical stress applied, except for the case when they are equal, for the wheel load located directly above the element. Owing to this fact, the duration of the principal stresses applied is also of a larger magnitude than the duration of the vertical stress applied, and the difference in these two durations increases with depth. Figures (3.8) and (3.9) were developed by Barksdale, and give suggested load duration times based on different vehicle speeds and depths, for both the principal and vertical stress pulses. The question arises as to which stress to use, and Barksdale suggests: (1) use the principal stress pulse for determining the dynamic modulus of elasticity, since principal stresses are applied in the triaxial cell, and (2) use the vertical stress pulse for the investigation of plastic properties and rutting, as these are related to the accumulation of vertical strain.

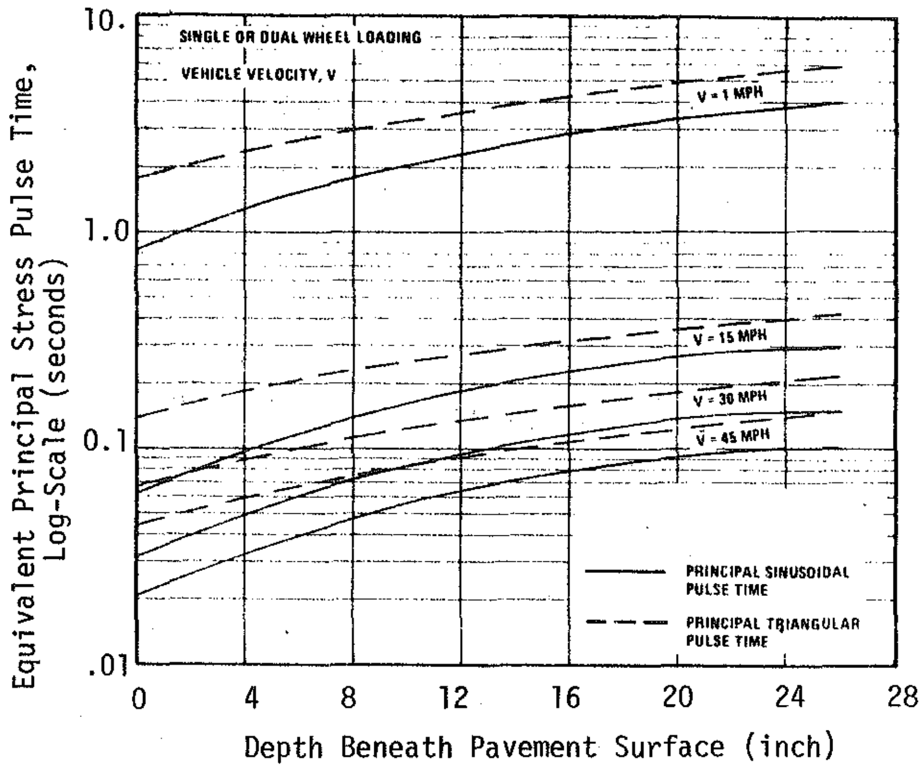


FIGURE 3.8 VARIATION OF EQUIVALENT PRINCIPAL STRESS PULSE TIME WITH VEHICLE VELOCITY AND DEPTH, (24).

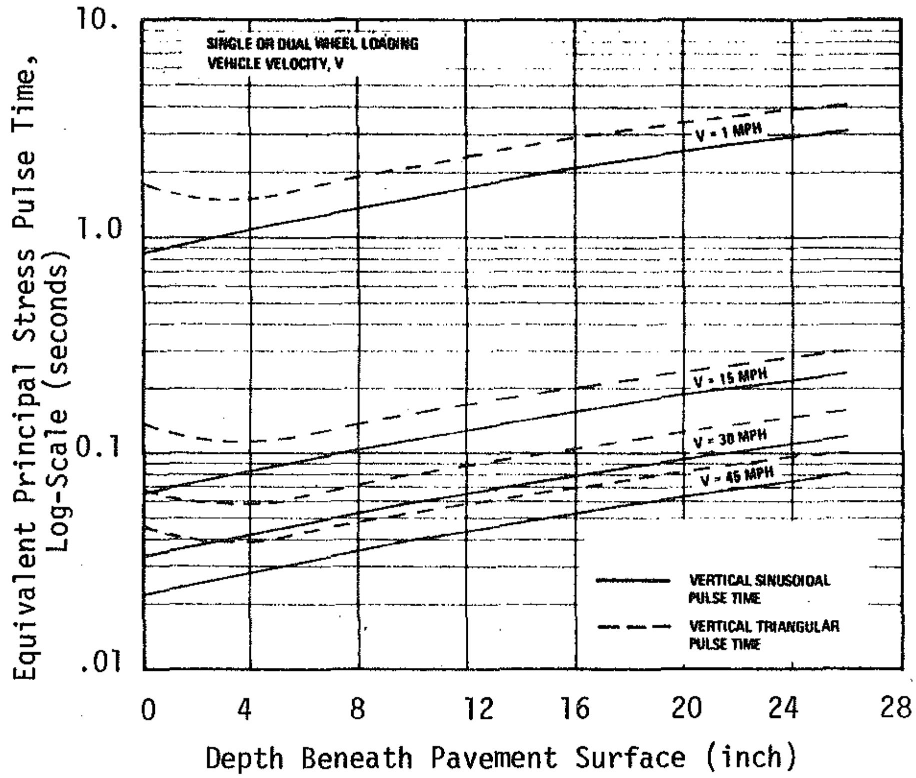


FIGURE 3.9 VARIATION OF EQUIVALENT VERTICAL STRESS PULSE TIME WITH VEHICLE VELOCITY AND DEPTH, (24).

Barksdale (24) also reported another finding in his work which bears mention here. For conventional flexible pavements, including those of deep-strength design, and for spring and summer temperatures, the load duration time is not affected by the pavement geometry or by layer stiffness and thickness. For engineering considerations, the effects of these are negligible.

In general, most repeated load triaxial tests are performed using a load duration of 0.1 second and a frequency of loading of 20 cycles per minute.

5.5. Confining Pressure

Just as the level of axial stress depends on load intensity and depth within the pavement structure, so does the confining or lateral pressure. Allen and Thompson (22) used non-linear finite element analyses for typical pavement sections to establish confining pressure values. Terrel and Awad (47) used a n-layer computer program to plot the variation of confining pressure as a result of moving load.

Recently, investigators have incorporated varying confining pressures which pulse in sequence with the axial load as shown in Figure (3.10). However, there is no general agreement as to the importance of such variation, since pulsating the confining pressure tends to overestimate the resilient properties of the specimen being tested.

6. TYPICAL VALUES OF TEST PARAMETERS

The various test parameters and criteria for choosing the values of test parameters have been presented. Typical values for these parameters are:

Load Frequency:	10 to 30 cpm
Load Duration:	0.04 to 0.25 seconds
No. of Repetitions:	10,000 to 100,000
Confining Pressure	0 to 25 psi
Deviator Stress:	1 to 70 psi
Sample Size:	1.4" x 3" to 4" x 8"
Load Wave Form:	square or sinusoidal

Table (3.1) lists specific values of these test parameters for past investigations, along with the range of resilient modulus values determined.

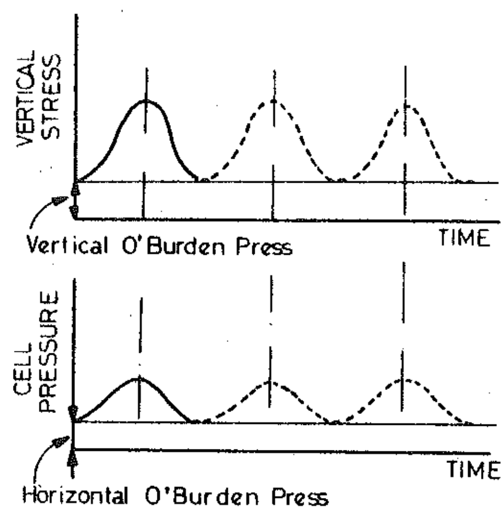


FIGURE 3.10 STRESS REGIME FOR THE REPEATED LOAD TRIAXIAL TEST, (41).

TABLE 3.1 SUMMARY OF LABORATORY TESTS TO EVALUATE THE ELASTIC PROPERTIES OF GRANULAR MATERIALS, (36):

Type of Rep. Loading	Reference	Material Investigated	Factors Investigated	Conf. Press. in p.s.i.	Deviator Stress in p.s.i.	Frequency	Number of Load Applications	Modulus of Resil. Deform. in p.s.i.
A. Triaxial repeated-loading compression tests	A. H. B. Seed and C. K. Chan (1961)	Silty sand	Effect of duration of stress applications	14.7	23.5 and 36.0 36.0 36.0	20/min. for 1/3 sec. 2 min. on, 2 min. off 20 min. on, 20 min. off	10,000	21,300 and 27,300 23,200 22,000
	B. Haynes (1961) Joint highway research project, Purdue University	AASHO base-course mat. Revised gradation, 3/4 in. max. size 1. Gravel 2. Crushed stone	- Percent of fines passing #200 mesh sieve: 6.2, 9.1 and 11.5% -Degree of saturation 70, 85 and 100%	15.0	55	40/min.	100,000	From 28,000 up to 63,000
	C. University of California (1962)	Niles aggregate, fairly rounded 3/4" max. size, 5% passing #200 sieve	-void ratio -confining pressure	1[illegible] and 2[illegible]	20, 40 and 60	20/min.	10,000	From 16,650 up to 54,500
	D. Jean Biarez ( )	Uniform sand - 0.016 in. in diam. Specimen dimensions 5.5 in. x 5.5 in. x 10 in.	Variation of E with the applied mean	Mean normal stress from 2.2 145 lbs./in.^{2}		Cyclic load - the rate of deformation is not indicated	of the order of 5	4,300 lbs./in.^{2} for mean =2.2 lbs./in^2 and 71,000 lbs./in.^{2} for mean = 145 lbs./in.^{2}
	E. D. H. Trollope, I. K. Lee, and J. Morris (1962)	poorly graded, (dry sand) (Earlston sand)	- Initial dry density (loose and dense) -Rate of deformation from 0.003 to .02 in./min. -Lateral pressure at constant stress -Effect of stress level at const. conf. press.	from 15.0 up to 45 p.s.i.	Stress level was varied	Cyclic load - rate of deformation from .003 to .02 in./min.	of the order of 100	From 35,000 up to 95,000 p.s.i.
	F. Texas Transp. Institute, W. A. Dunlap (1963)	Graded material - 1 in. max. size - 6% passing #200 - P.I.S., L.L. 21 Folding w.c. 5.5%	Variation of modulus of resil. def. with confin. press.	From 3 up to 30 p.s.i.	34.5 and 51.[illegible] p.s.i.	30/min. - duration 0.2 sec.	130,000 reps	from 3[illegible],000 up to 160,000 psi

CHAPTER 4

FACTORS AFFECTING THE RESILIENT RESPONSE

1. COHESIVE SOILS

Unlike other materials which will be discussed, cohesive subgrade materials cannot be accurately characterized without great attention being given to the preparation of the sample. In determining the resilient parameters for clays, the lab samples should be identical in composition to the field. This means that water content, density and the structural arrangement of the particles (which is controlled by the method of compaction used in preparing the sample) must be identical. In this section the effects of the most important soil and test parameters on the repeated load characteristics of clay soils will be discussed.

1.1 Number of Stress Applications

Silt and clay subgrade materials generally exhibit a stiffening behavior with an increasing number of stress applications, N. The total deformation of test specimens increases with increasing N, and the resilient deformation tends to decrease. Most investigators tend to evaluate the resilient properties based on sample response after a relatively small number of applications, of the order of 5,000 or less, and this can present a misleading picture of the resilient behavior.

In tests on stiff clays, Dehlen (75) found that 1,000 stress repetitions were sufficient to condition the sample for testing without significantly altering the specimen response. Conditioning the sample helps to avoid variations in axial deformations caused by end imperfections. He found that once the sample was conditioned, the response obtained at a relatively low number of stress applications (N = 50 to 100) was representative. As long as N was small, testing at many different stress levels was possible, before stiffening behavior became significant. With the number of stress applications on the order of 25,000, the test specimens stiffen and the response is affected, but at N = 100, many different stress levels could be applied, and the resilient properties at these levels could be determined. Tanimoto and Nishi (46) also emphasize the importance of selecting the proper number of stress applications at which to determine resilient properties.

Seed et al (6) also found that the response of clay samples was dependent on the number of stress applications. In general, they found that compacted clays develop their greatest resilient deformation when N is less than 5000. This resilient deformation was found to decrease significantly at $N > 100,000$. Permanent deformations continued to increase at this number of applications. Figure (4.1) shows the effect of N on permanent strain with different levels of stress. Larger stresses took fewer load applications to yield excessive permanent strains.

Some question remains as to what number of stress applications is appropriate for the determination of resilient properties. The resilient behavior of cohesive soils is only evident at small numbers of load application; however, these soils are subjected to many more load applications in the field. It appears that the determination of the resilient modulus at lower numbers of stress applications is a conservative measure. It is lowest when resilience is greatest, and increases as the sample stiffens at high numbers of load applications. With N of the order of that expected in the field, the resilient modulus is much greater due to the subsequent stiffening of the soil, meaning increased subgrade support.

1.2. Thixotropy

Investigators have found that the response of cohesive soils can be greatly influenced by the length of time between preparation and testing. The strength increases as the time between preparation and testing (storage time) increased. However, this effect tends to diminish as the number of load applications increased.

Seed et al (6) found the resilient deformation decreased (the resilient modulus increased) as the time between compaction and testing increased. This effect could be seen for $N < 40,000$, but for $N > 40,000$, samples of all different ages began to exhibit the same behavior. For a number of load applications of the order of 10, the resilient modulus for 1 day and 50 days storage time may differ by as much as 300 or 400%. Figure (4.2) shows the effect of different storage times on the resilient modulus for a range of number of stress applications. For large value of N, the effects of aging are reduced and the same results are obtained for samples tested immediately after compaction as those tested

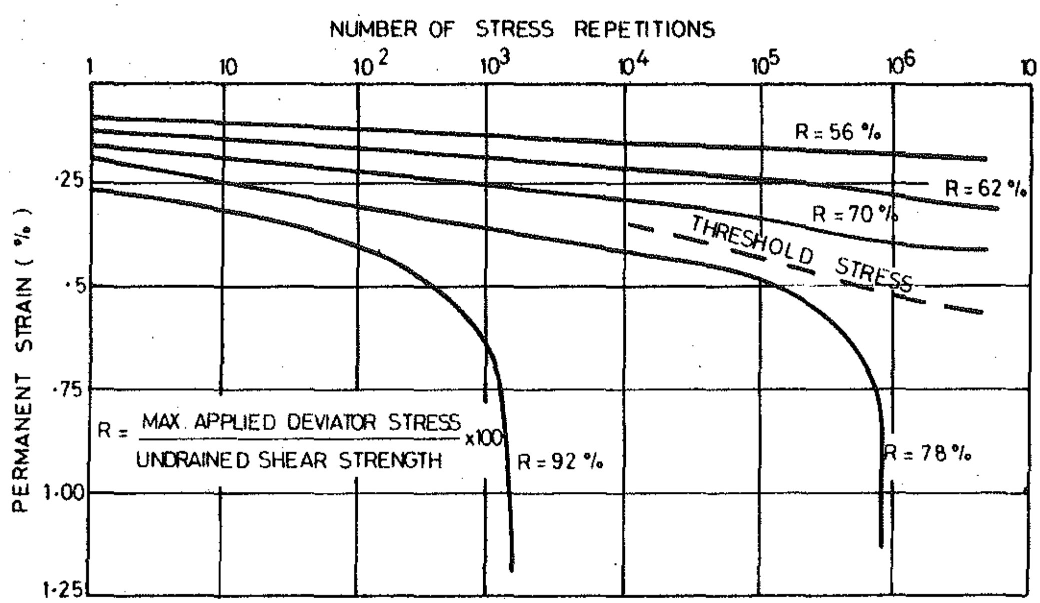


FIGURE 4.1 PERMANENT STRAIN Vs. NUMBER OF LOAD REPETITIONS FOR A SATURATED SILTY CLAY, (41).

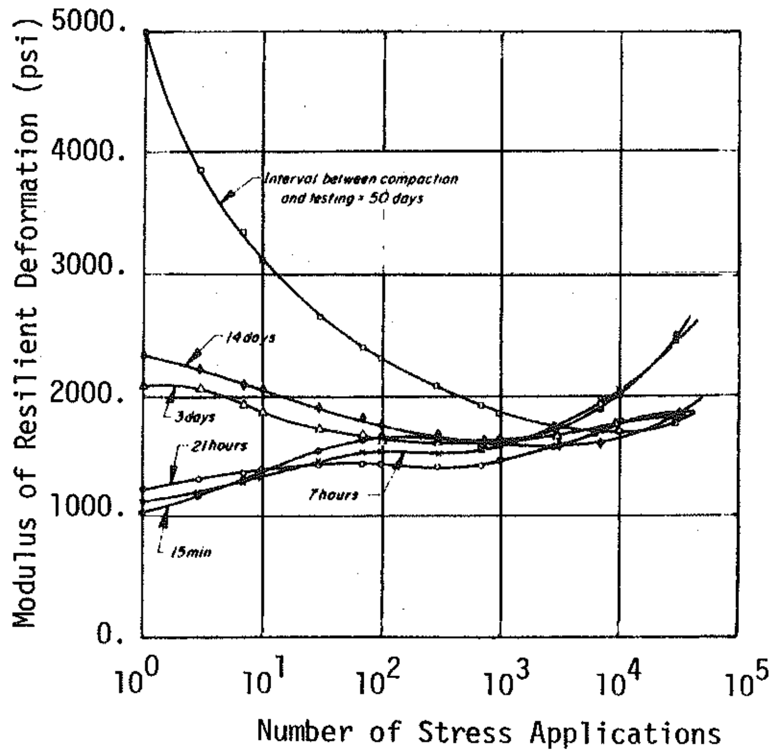


FIGURE 4.2 EFFECT OF THIXOTROPY ON RESILIENCE CHARACTERISTICS, AASHO ROADTEST SUBGRADE SOIL, (6).

after a period of time. Tanimoto and Nishi (46) also found this to be the case, but water content appeared to affect the thixotropic strength gain. At water contents far below or well above the optimum, they found that storage time had little effect on the specimen response. However, at water contents just above optimum this effect is much more pronounced. Again, these effects were destroyed by high numbers of stress applications. Figure (4.3) illustrates this point for a silty clay with an optimum water content of about 18 percent.

The effect of storage time on strength is still uncertain. The number of stress applications used in the lab can be developed usually within one day, whereas the number of stress applications under in-service conditions may take many years to develop. Once again, it appears that lab estimates of strength are conservative due to the much shorter times involved.

1.3 Stress Intensity

In all investigations, the relationship between the resilient modulus and the deviator stress is similar. At low stress levels, the resilient modulus decreases and the deviator stress increases. This is true up to a deviator stress of about 10 psi where the resilient modulus is found to be unaffected or increases only slightly with further increase in deviator stress. Because of this dependence on the deviator stress, it is important that lab tests are conducted at stresses which are expected in the field. Figure (4.4) shows the decrease in M_R as the deviator stress increases from 2 to 10 psi under a constant radial pressure. It also shows that Poisson's ratio is only slightly affected by changes in the deviator stress.

For test on silty clays Mitchell et al (35), using 24,000 load applications, found that the resilient modulus decreased with increasing deviator stress up to 25 psi, above which the resilient modulus increased slightly. Seed et al (6) had also found that the resilient modulus decreased rapidly with a variation of 300 to 400 percent as the deviator stress increased from 3 to 15 psi. Above this range the resilient modulus was observed to increase slightly, as shown in Figure (4.5). This means that as the depth of a soil element

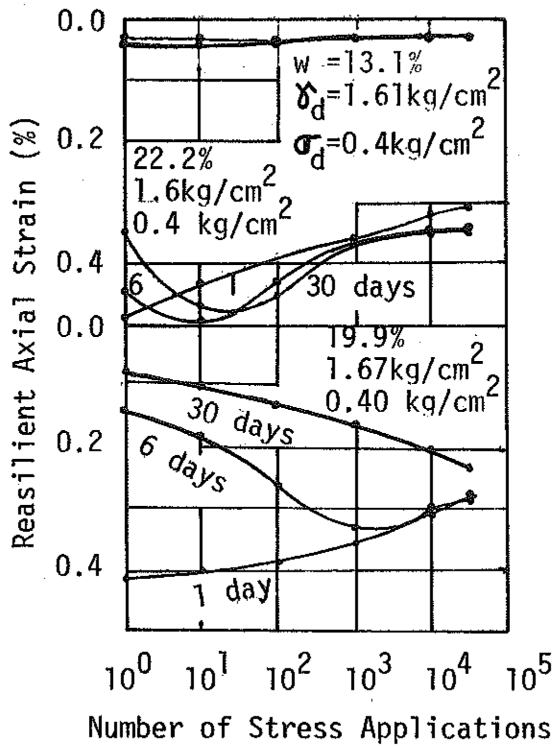


FIGURE 4.3 EFFECT OF STORAGE PERIOD ON RESILIENCE CHARACTERISTICS OF COMPACTED SUBGRADE MATERIAL,(46).

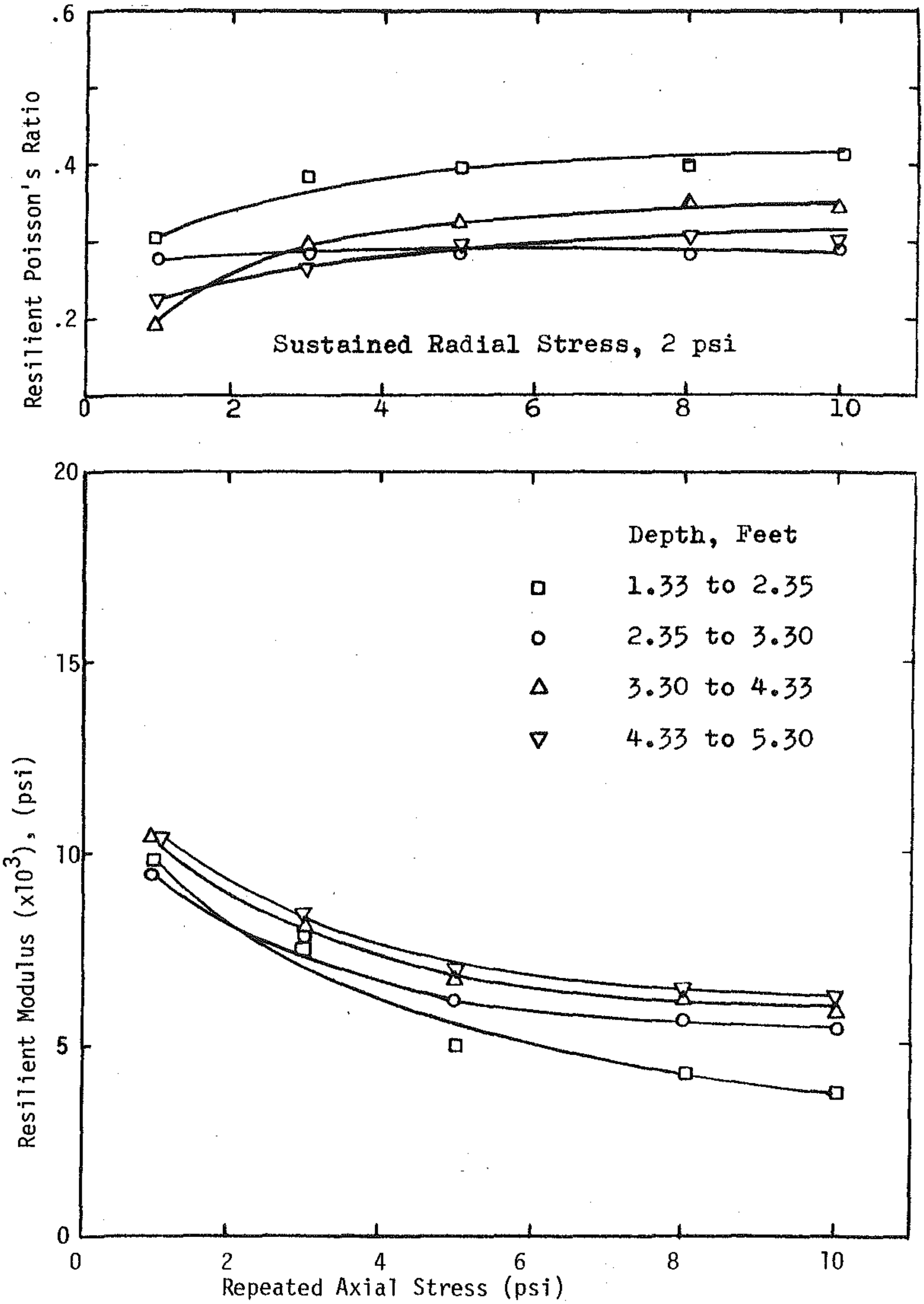


FIGURE 4.4 SECANT MODULUS AND POISSON'S RATIO OF CLAY SUBGRADE AS A FUNCTION OF REPEATED AXIAL STRESS AND DEPTH BENEATH PAVEMENT SURFACE, (29).

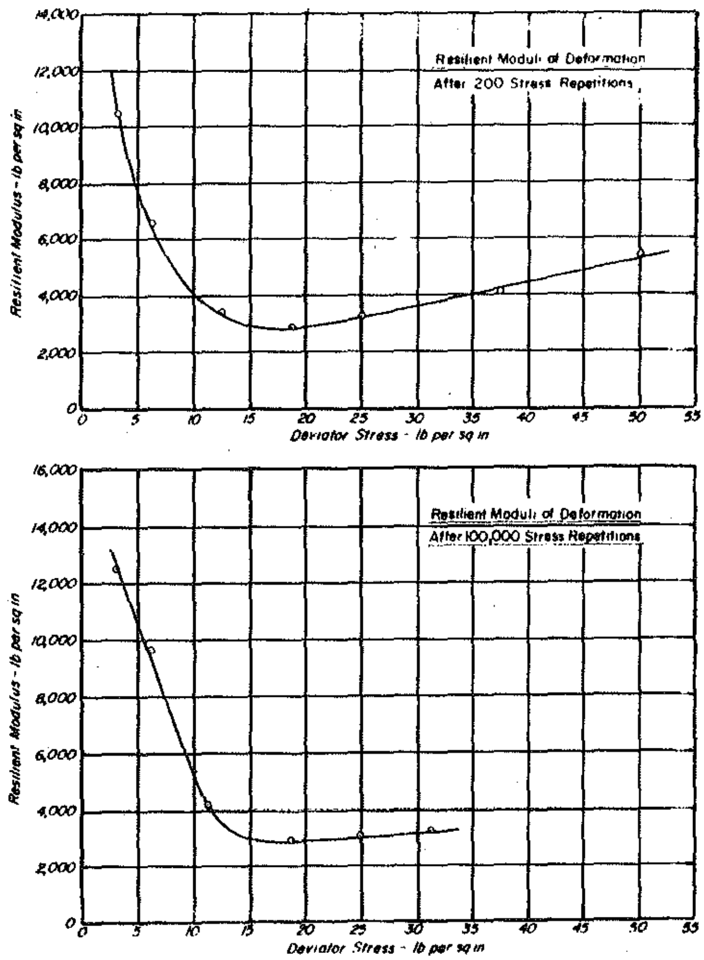


FIGURE 4.5 EFFECT OF STRESS INTENSITY ON RESILIENCE CHARACTERISTICS FOR AASHO ROAD TEST SUBGRADE SOIL, (6).

increases, and the applied deviator stress decreases, there will be an increasing resilient modulus with depth, assuming a uniform soil. Seed et al (43), using repeated plate load tests, and Tanimoto and Nishi (46) have also determined the same relationship.

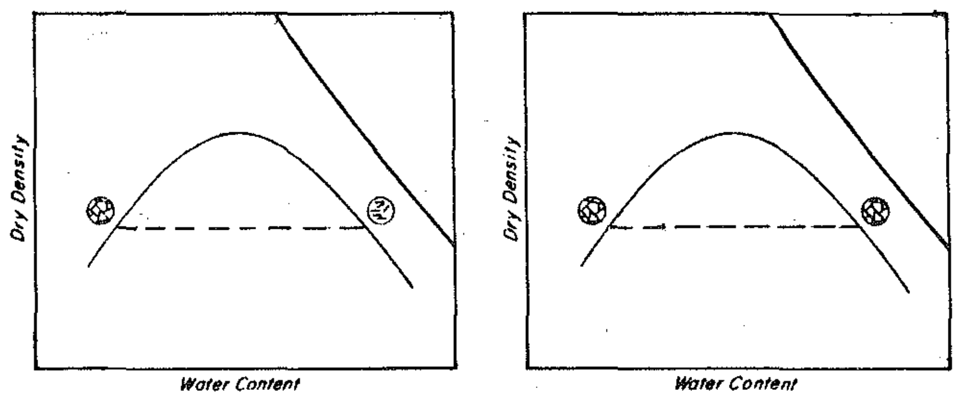
1.4. Method of Compaction

The method of compaction employed during the preparation of a cohesive soil test specimen has a profound effect on the particle structure and subsequent behavior. Changes in particle structure are related to the shear strain induced in the soil by different methods of compaction.

When cohesive soils are compacted to relatively low degrees of saturation, the shear strain induced in the soil by any method of compaction is not appreciable. Particles assume a random edge to face configuration, which is termed a flocculated structure. The behavior of these samples at low degrees of saturation is similar, no matter what method of compaction is employed (76, 78, 81).

At higher degrees of saturation, such as those on the wet side of the optimum water content, the shear strain induced by various compaction methods may vary greatly. As the hammer or tamping foot penetrates, the soil tends to heave upward around it. The particles tend to align themselves parallel with the surface of shear and with one another. Throughout the sample there are local areas where the particles are situated predominately parallel to each other, termed a dispersed structure (81, 82, 83, 95).

For high degrees of saturation and a static method of compaction, a flocculated structure is retained. Because pressure is applied to the entire surface of the soil, no shear strain, which causes the dispersed structure, is induced. We can obtain the same flocculated structure for a high degree of saturation by soaking the sample. The boundary between the higher and lower degree of saturation appears to be at a degree of saturation of approximately 85 percent. This closely corresponds to the line of optimum water contents. Figure (4.6) shows the different particle structures resulting from kneading and static compaction at different degrees of saturation.



a) Kneading Compaction   b) Static Compaction

FIGURE 4.6 PARTICLE ORIENTATIONS IN COMPACTED CLAYS, (6).

For samples compacted dry of optimum, the resilient behavior is essentially the same, regardless of the method of compaction. The resilient behavior of samples compacted wet of optimum varies greatly, depending upon the compaction method. Flocculated structures, obtained by static compaction, produce higher values of resilient modulus and lower resilient deformations than for the dispersed structures obtained by kneading compaction. A comparison of the resilient properties is shown in Figure (4.7) for a degree of saturation of 95 percent.

Using the appropriate method of compaction in preparation, it is possible to simulate field conditions very closely. Seed et al (6) concluded that the particle structures induced by rubber tired rollers in the field, and kneading compaction in the lab, were very similar, since samples from both exhibited similar properties. Clays in the field are typically compacted dry of optimum and thus retain a flocculated structure. To test a critical condition, such as a high degree of saturation, the particle structure tested in the lab must also possess a flocculated structure. This can be obtained in the two ways pointed out above: kneading compaction at a low degree of saturation, and subsequent soaking, or static compaction at the desired degree of saturation. To soak the former would require a great deal of time, whereas static compaction can obtain the required results in much less time.

1.5. <u>Compaction Density and Water Content</u>

All investigators have found that an increasing water content at compaction leads to an increase in resilient deformation, and a decrease in strength and resilient modulus. For a given compactive effort, the resilient deformation is relatively low at water contents dry of optimum, but it increases rapidly as the water content at compaction exceeds the optimum. Seed et al (6) found that for a given dry density, the resilient modulus decreased as the water content at compaction increased. The resilient deformations increased with the water content. Seed et al (43) and Tanimoto and Nishi (46) reported the same results. Figure (4.8), from Monismith and Finn (38), relates the resilient modulus to water content and dry density. It

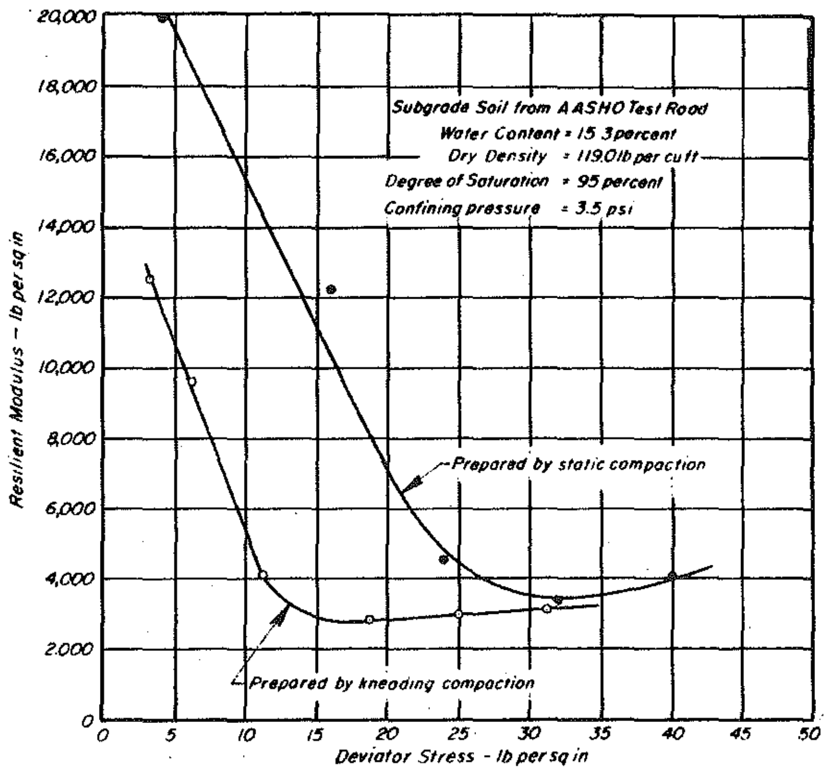


FIGURE 4.7 EFFECT OF METHOD OF COMPACTION ON RELATIONSHIP BETWEEN RESILIENT MODULUS AND STRESS INTENSITY, (6).

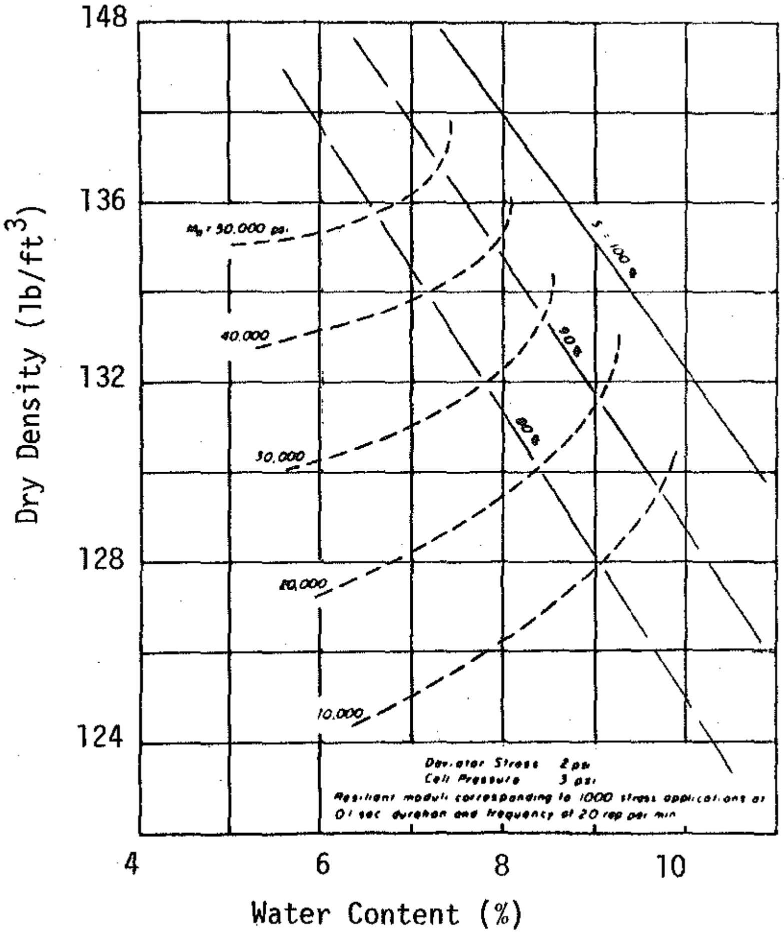


FIGURE 4.8 WATER CONTENT-DRY DENSITY-RESILIENT MODULUS RELATIONSHIP FOR SUBGRADE SOIL, (38).

shows the decrease of M_R with increasing water content. It also shows that for a given water content at compaction, as the dry density increases, the resilient modulus also increases, until it levels off at the optimum condition, then M_R begins to decrease slightly.

At high degrees of saturation, minor changes in dry density or water content have significant effects on the resilient behavior. Seed suggested that this is attributable to the marked change which can take place in the soil structure at this range. He feels that it is desirable to compact samples to a saturation of about 80 percent to avoid this and minimize the resilient deformation. One further caution is also made. Under field conditions, traffic loading of the subgrade soil may tend to densify it, and also reduce the water content. Both of these conditions, along with the large number of repeated loadings, will lead to higher strength and resilient modulus than expected. This is an important consideration in pavement deflection predictions.

Figure (4.9) shows that as the dry density increases, the resilience decreases. If two samples are allowed to absorb water to a degree of saturation of 90% after compaction at an identically lower degree of saturation, the one compacted to a higher density will exhibit much less resilience as shown in this figure. Figure (4.10) shows that the resilient deformation of samples soaked to a higher degree of saturation after being compacted at a low degree of saturation is much smaller than those samples compacted by kneading to the same final condition. The resilient deformation of samples compacted directly to a high degree of saturation may be many times larger than those which attain the same degree of saturation by soaking after compaction at a lower water content.

During construction, a subgrade will most often be compacted to a degree of saturation of approximately 75 percent. This would correspond to a flocculated particle structure as stated previously. After a long period of time, the subgrade may absorb water with no volume change, raising its degree of saturation to about 90 or 95 percent. It is virtually impossible to reproduce this condition by soaking, because the degree of saturation will not be uniform throughout

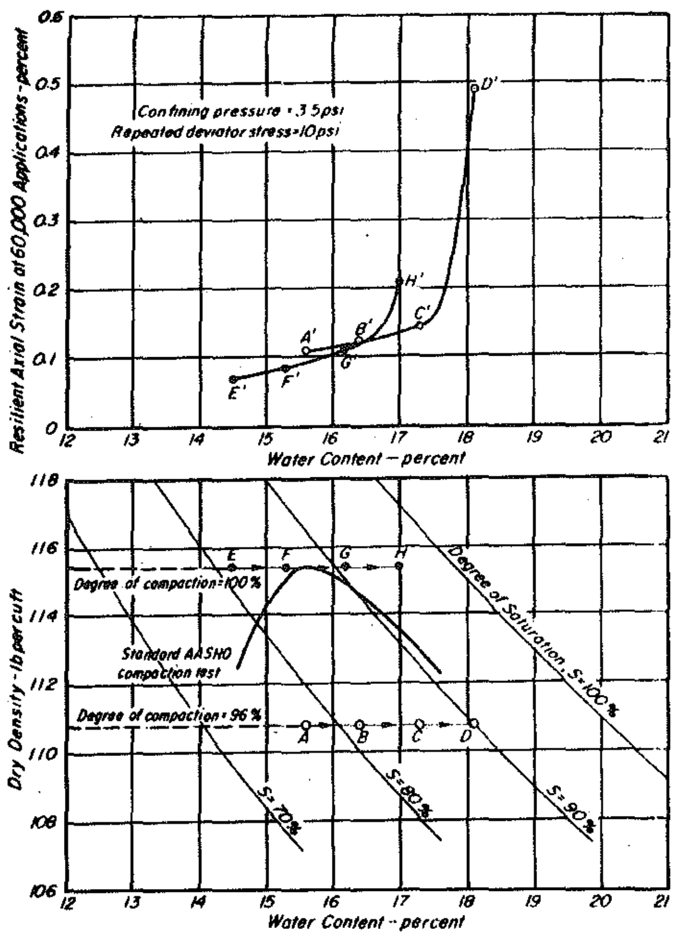


FIGURE 4.9 EFFECT OF INCREASE IN WATER CONTENT AFTER COMPACTION ON RESILIENT DEFORMATIONS, AASHO ROAD TEST SUBGRADE SOIL, (6).

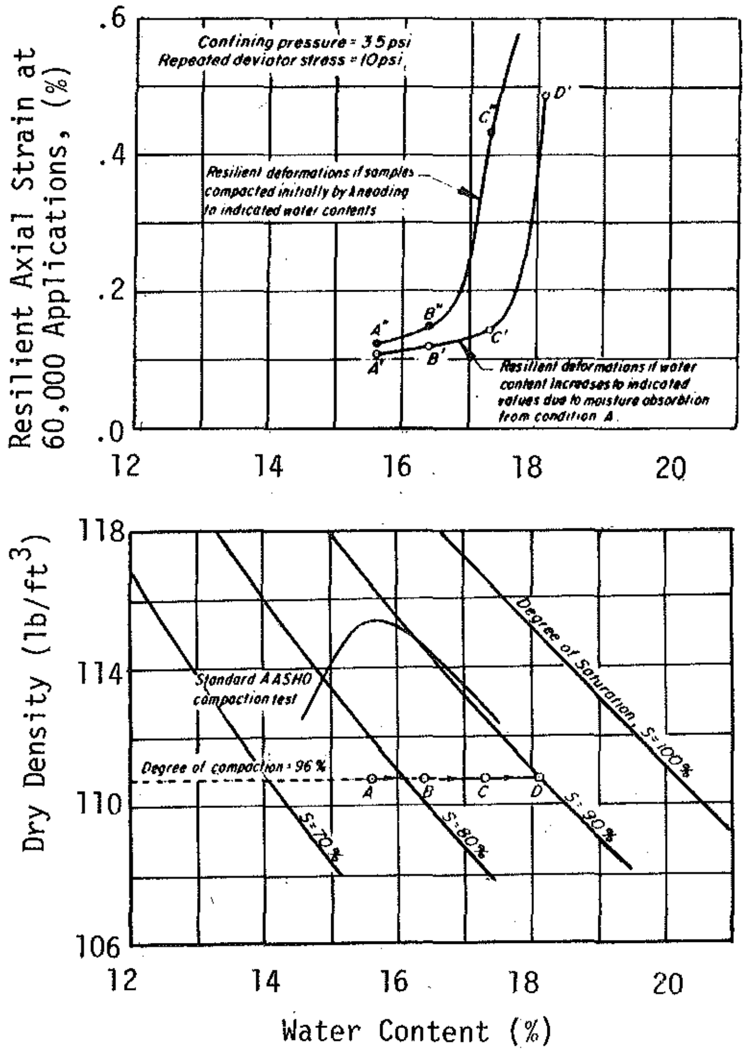


FIGURE 4.10 EFFECT OF METHOD OF ATTAINING FINAL MOISTURE CONDITION ON RESILIENT STRAINS, (6).

the sample. The exterior portions may be saturated 100 percent, while the center may still be only about 80 percent. This is the reason static compaction is used for tests on samples with degrees of saturation greater than 85 percent.

1.6. Confining Pressure

The resilient response of cohesive soils is relatively unaffected by changes in cell pressure during the repeated load triaxial test. In tests on subgrade soils from a prototype pavement, Hicks (28) reported that the stress-strain relationship is little affected by changes in radial stress. This was typical of all samples tested. Table (4.1) illustrates these findings. In tests on silty clays, for the same repeated load, Tanimoto and Nishi (46) reported that the resilient modulus is unaffected by confining pressure, as shown in Figure (4.11). Terrel and Awad (47) also reported similar findings.

1.7. Stress Sequence

Dehlen (75) studied the effects of stress repetitions and sequence on stiff silty clay soil. He found that if 25,000 repetitions were applied at each stress level, the sequence in which the stress levels were applied had a significant effect on the measured resilient modulus, since the sample tended to stiffen due to prior applications of stress. However, if only 100 repetitions were applied at each stress level, the stress sequence had little effect on the resilient modulus measured, provided that the stresses applied were in the range expected under a pavement. In both the 100 and 25,000 repetition tests, Poisson's ratio was relatively unaffected by stress sequence.

2. COHESIONLESS SOILS

The behavior of granular materials found in the base and subbase courses of flexible pavements differs greatly from that of fine-grained soils found in the subgrade. The factors which affect their behavior are more numerous and most of these are related to conditions under which these soils are tested. The state of stress during testing appears to be of much greater importance, and the method of sample preparation seems to be of less importance than previously seen for cohesive soils. In this section, the soil properties and triaxial test parameters which significantly affect the response of cohesionless soils during testing are discussed.

TABLE 4.1 STRESS-STRAIN PAIRS FOR SUBGRADE SAMPLE NO. 2-1, (29).

Repeated Axial Stress	Sustained Radial Stress	Axial Micro Strain	Radial Micro Strain
10.0	3.0	2900.0	1150.0
8.0	3.0	1850.0	750.0
5.0	3.0	900.0	362.0
3.0	3.0	380.0	150.0
1.0	3.0	85.0	26.0
10.0	2.0	2700.0	1100.0
8.0	2.0	1850.0	750.0
5.0	2.0	975.0	387.0
3.0	2.0	390.0	150.0
1.0	2.0	110.0	33.7
10.0	1.0	2700.0	1100.0
8.0	1.0	1850.0	750.0
5.0	1.0	975.0	400.0
3.0	1.0	430.0	168.7
1.0	1.0	95.0	30.0

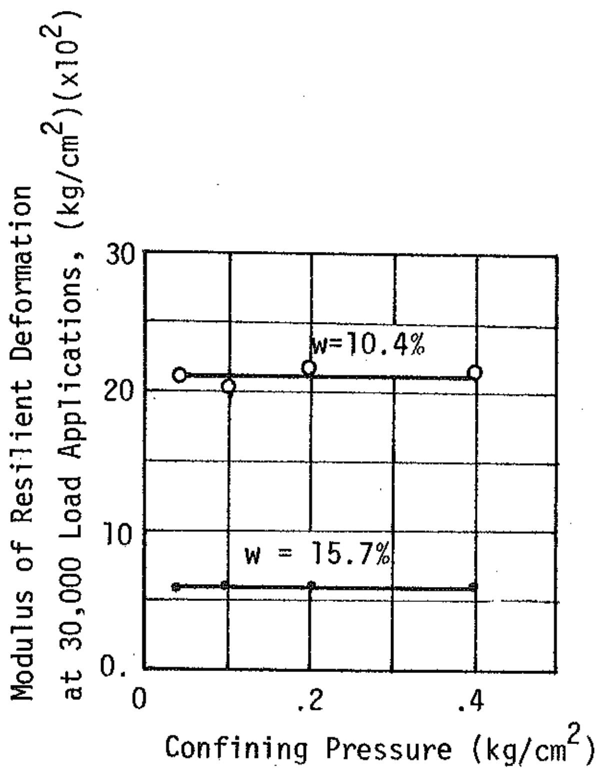


FIGURE 4.11 EFFECT OF CONFINING PRESSURE ON RESILIENCE CHARACTERISTICS OF COMPACTED SUBGRADE MATERIAL, (46)

2.1. Number of Stress Applications

Although researchers are not in full agreement as to the type of effect the number of stress applications has upon the resilient response, they are in agreement that the magnitude of this effect is slight. Some have found that the resilient modulus increases slightly, while still others have observed a decrease in resilient modulus with increasing load applications. Morgan (39) in tests on fine sands, reported that the resilient modulus increases slightly up to about 10,000 load applications, whereafter it remained constant. Tanimoto and Nishi (46) reported similar results, with resilient strain decreasing and resilient modulus increasing slightly, with increasing number of stress applications.

Between 100 and 25,000 stress applications, Hicks (29) found that the values of both the resilient modulus and Poisson's ratio remained fairly constant for dry granular materials. The lower limit of this range was raised slightly for partially saturated materials, where these properties were constant beyond about 100 to 300 stress applications. For saturated granular materials, these properties are constant up to approximately 1,000 stress applications, beyond which the resilient modulus decreases slightly and Poisson's ratio increases slightly. Hicks suggested that this is due to the build-up of pore water pressure and a corresponding decrease in effective confining stress.

Hicks and Monismith (27), and Barksdale and Hicks (23) found that approximately 1,000 stress applications will properly condition the sample and avoid variations in the axial strain due to end imperfections. Once the sample is conditioned, 50 to 100 stress applications can be used to properly characterize the resilient response. They also stated that one sample can be used in this manner to determine the resilient response for many different stress intensities, provided they are in the range of those stresses likely to occur in the pavement. The fact that one sample can be used to study the resilient response under various stress intensities illustrates that a complex stress history has a little effect on the resilient response. For saturated granular materials, they found that the sample response was subject to change due to the build-up of pore water pressure which causes a reduction in the effective confining stress. This appears to be related to the phenomenon of liquefaction. Studies showed that the possibility of this occurrence was reduced if these samples were conditioned in a drained state.

Kalcheff and Hicks (31) reported that the number of stress applications had little affect on the resilient response of granular materials. For those samples with complex stress histories, they recommended 150 to 200 stress applications to get a good estimate of the resilient properties.

2.2. Stress Intensity

Once again, researchers have failed to find unanimity on the effect the deviator stress on the resilient modulus; however, they do agree that the effect (whatever it is) is slight. Hicks (29) stated that all studies indicate that the resilient modulus is relatively unaffected by the magnitude of the deviator stress. The investigations of Trollope (59), Mitchell (35), Kallas and Riley (63) and Seed et al (43) all came to this conclusion. Morgan (39) found that the resilient modulus decreases and the permanent deformation increases with increasing deviator stress, Figure (4.12). Also, he reported that for a range in deviator stress from 20 to 50 psi, Poisson's ratio appear to be constant.

Hicks (29) found that for lower levels of axial stress, a slight softening occurred in the axial strain, whereas at high levels of axial stress, the specimens tended to stiffen. A softening pattern was always observed for the radial strains. These points are illustrated by the data in Table (4.2). Figure (4.13) shows the variation of axial and radial strains with axial stress. Hicks also found that Poisson's ratio always increased with increasing deviator stress or principal stress ratio, but this increase appeared to be random.

Hicks and Monismith (27) reported a slight increase in resilient modulus with increasing deviator stress (principal stress ratio), as shown in Figure (4.14). Figure (4.15) shows this increase in Poisson's ratio with deviator stress. Hicks and Monismith also point out that Poisson's ratio can be reasonably approximated by the following equation:

$$\nu_r = A_0 + A_1 \left(\frac{\sigma_1}{\sigma_3}\right) + A_2 \left(\frac{\sigma_1}{\sigma_3}\right)^2 + A_3\left(\frac{\sigma_1}{\sigma_3}\right)^3 \qquad (4.1)$$

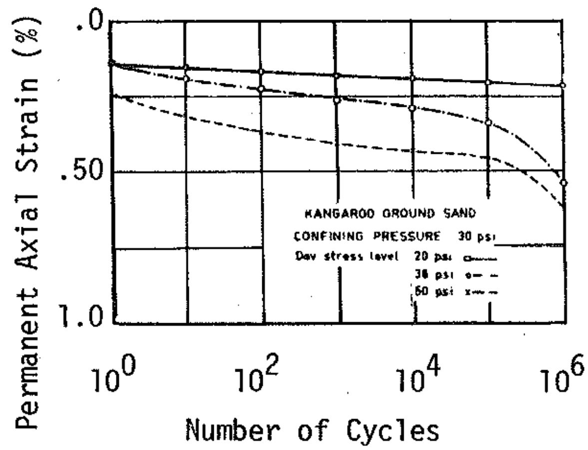


FIGURE 4.12 VARIATION OF PERMANENT AXIAL STRAINS WITH NUMBER OF CYCLES AND DEVIATOR STRESS AT CONSTANT CONFINING PRESSURE, (39).

TABLE 4.2 STRESS-STRAIN PAIRS FOR DRY COARSE AGGREGATE SAMPLE, (29).

REPEATED AXIAL STRESS	SUSTAINED RADIAL STRESS	AXIAL MICRO STRAIN	RADIAL MICRO STRAIN
35.	50.	400.0	102.5
30.	50.	337.5	81.2
20.	50.	235.0	45.0
15.	50.	170.0	23.7
10.	50.	110.0	12.5
35.	30.	550.0	170.0
30.	30.	475.0	145.0
20.	30.	325.0	82.5
15.	30.	245.0	52.5
10.	30.	160.0	25.0
35.	20.	675.0	262.5
30	20.	637.5	235.0
20.	20.	440.0	127.5
15.	20.	335.0	78.7
10.	20.	205.0	41.2
5.	20.	82.5	7.5
35.	10.	1150.0	625.0
30.	10.	1012.5	506.2
20.	10.	712.5	293.7
15.	10.	550.0	195.0
10.	10.	390.0	130.0
5.	10.	150.0	38.7
30.	5.	1325.0	1100.0
20.	5.	1062.5	687.5
15.	5.	825.0	431.2
10.	5.	550.0	237.5
5.	5.	280.0	97.5
3.	5.	145.0	35.0
15.	3.	850.0	562.5
10.	3.	612.5	337.5
5.	3.	350.0	165.0
3.	3.	202.5	80.0
10.	2.	724.0	580.0
5.	2.	435.0	269.0
3.	2.	275.0	145.0
5.	1.	487.5	481.2
4.	1.	420.0	381.2
3.	1.	342.0	293.7

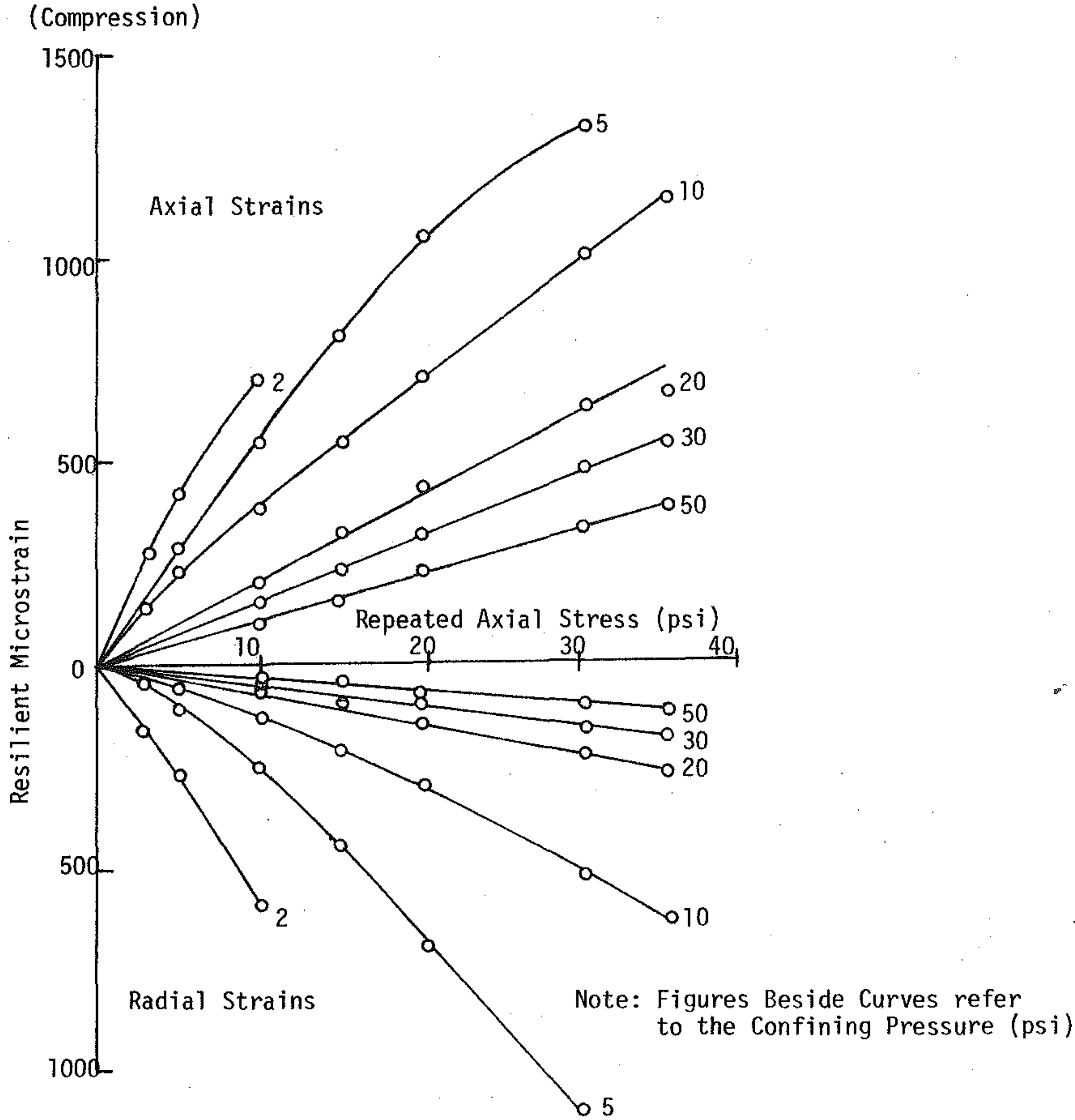


FIGURE 4.13 VARIATION IN AXIAL AND RADIAL STRAINS WITH AXIAL STRESS,(27).

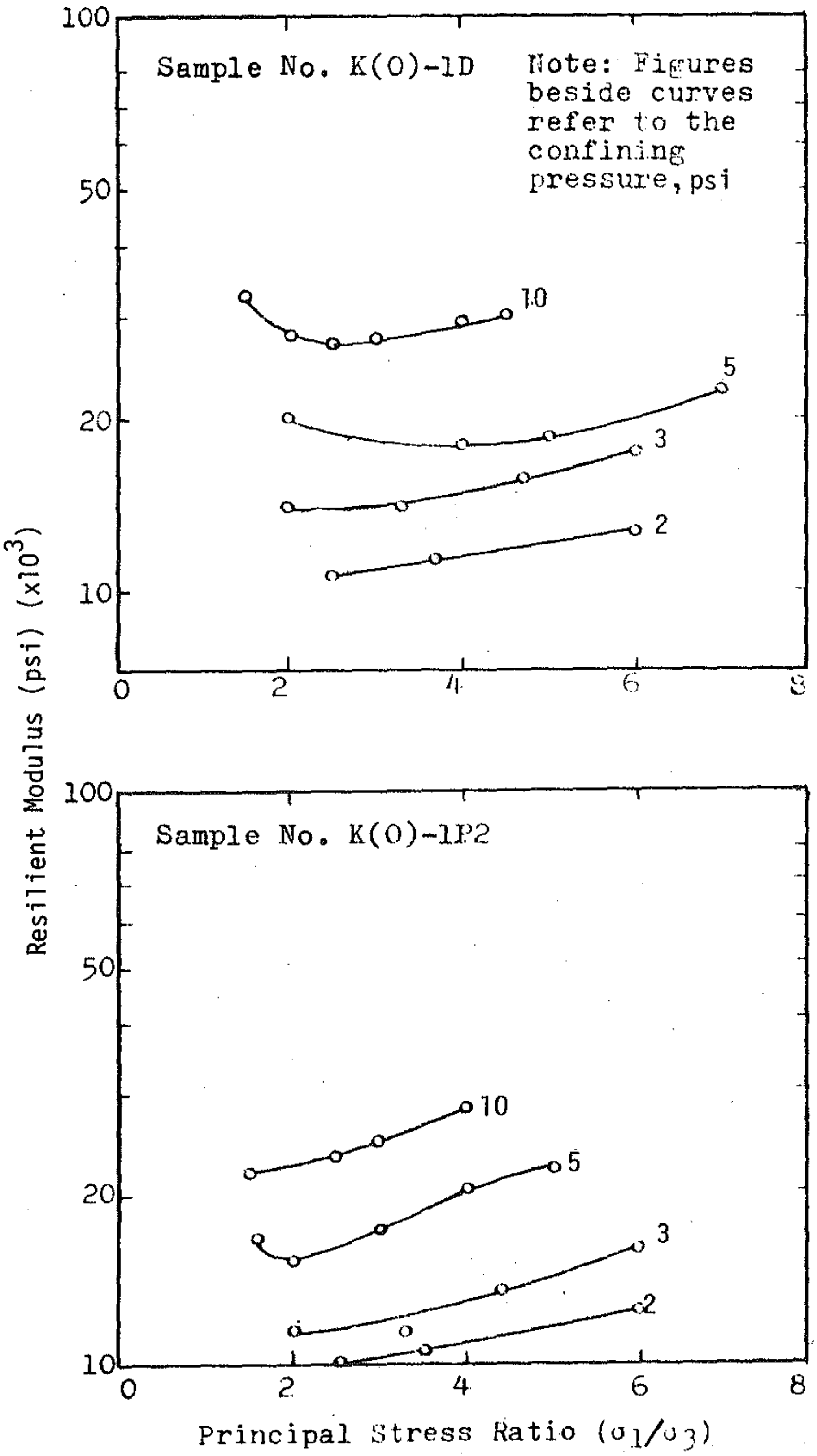


FIGURE 4.14 VARIATION IN RESILIENT MODULUS WITH PRINCIPAL STRESS RATIO, (27).

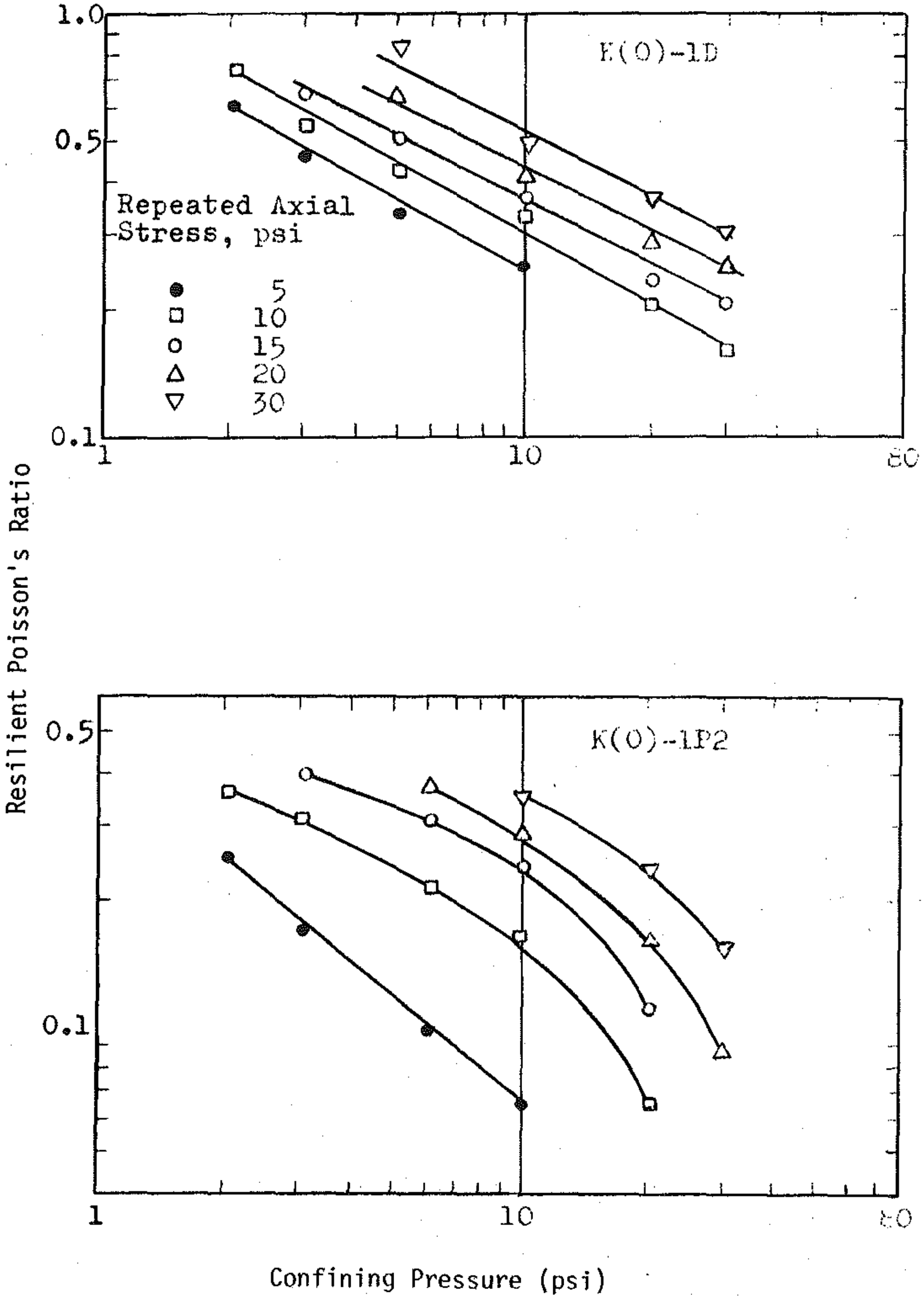


FIGURE 4.15 VARIATION IN RESILIENT POISSON'S RATIO WITH STRESS LEVEL, (27).

Barksdale and Hicks (23) in studies on plastic strain in sands found that at lower values of deviator stress, the rate of accumulation of plastic strain tended to decrease as the number of load applications increased. At higher values of deviator stress, the reverse was found to be true: the rate of accumulation of plastic strain increased as the number of stress applications increased. This is significant in studies of rutting in flexible pavements.

2.3. Stress Sequence

One specimen could be used to test the resilient response of sand over a wide range of stress levels which could be applied in any order without error (27, 31, 22). However, Kalcheff and Hicks (31) reported that the measured plastic properties changed considerably as the stress sequence varied.

2.4. Confining Pressure

There seems to be no question as to the effect of confining pressure, in a triaxial cell, upon the resilient modulus. The higher the confining pressure, the higher the resilient modulus (23, 29, 38, 39, 47, and 61). Tests at the Texas Transportation Institute in 1963 by Dunlap (61) show that the resilient modulus increased by 500% as the confining pressure increased from 3 to 30 psi with the largest increase occurring at confining pressures from 1 to 10 psi. Direct relationships, as introduced earlier, were suggested between the resilient modulus M_R and either of the confining pressure σ_3 or the sum of the principal stresses, θ.

$$M_r = K\,\sigma_3^{\,n} \tag{4.2}$$

$$M_r = K^1\,\theta^{n_1} \tag{4.3}$$

in which K, K^1, n and n^1 are constants determined by a least squares curve fitting method. Figures (4.16), (4.17), (4.18), (4.19), and (4.20) show these relationships for sand, dry gravel and base course aggregates. Morgan (39) attempted to explain these relationships by the elastic compression of the soil skeleton. He stated that as the confining pressure increased, the side portions of the sample would be held more firmly in place. Consequently, the soil resilience will be reduced. He also

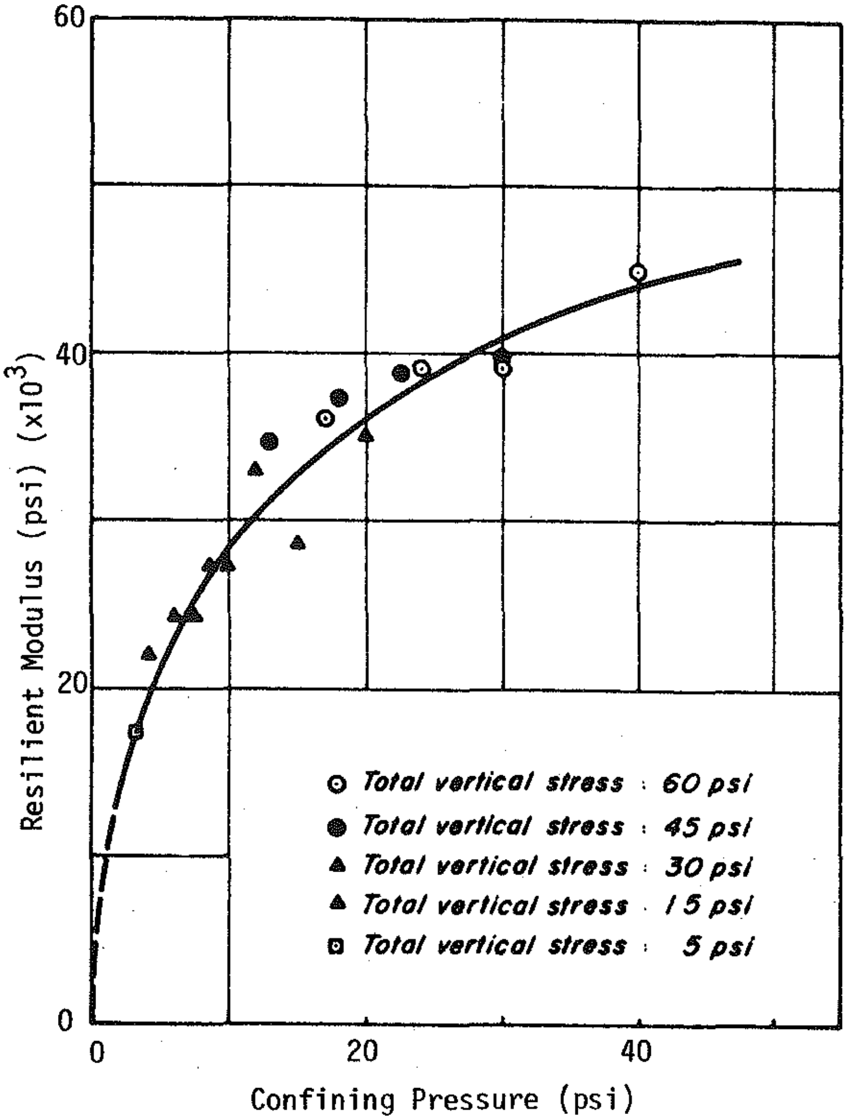


FIGURE 4.16 ARITHMETIC PLOT OF THE RELATIONSHIP BETWEEN RESILIENT MODULUS AND CONFINING PRESSURE FOR SAND, (43).

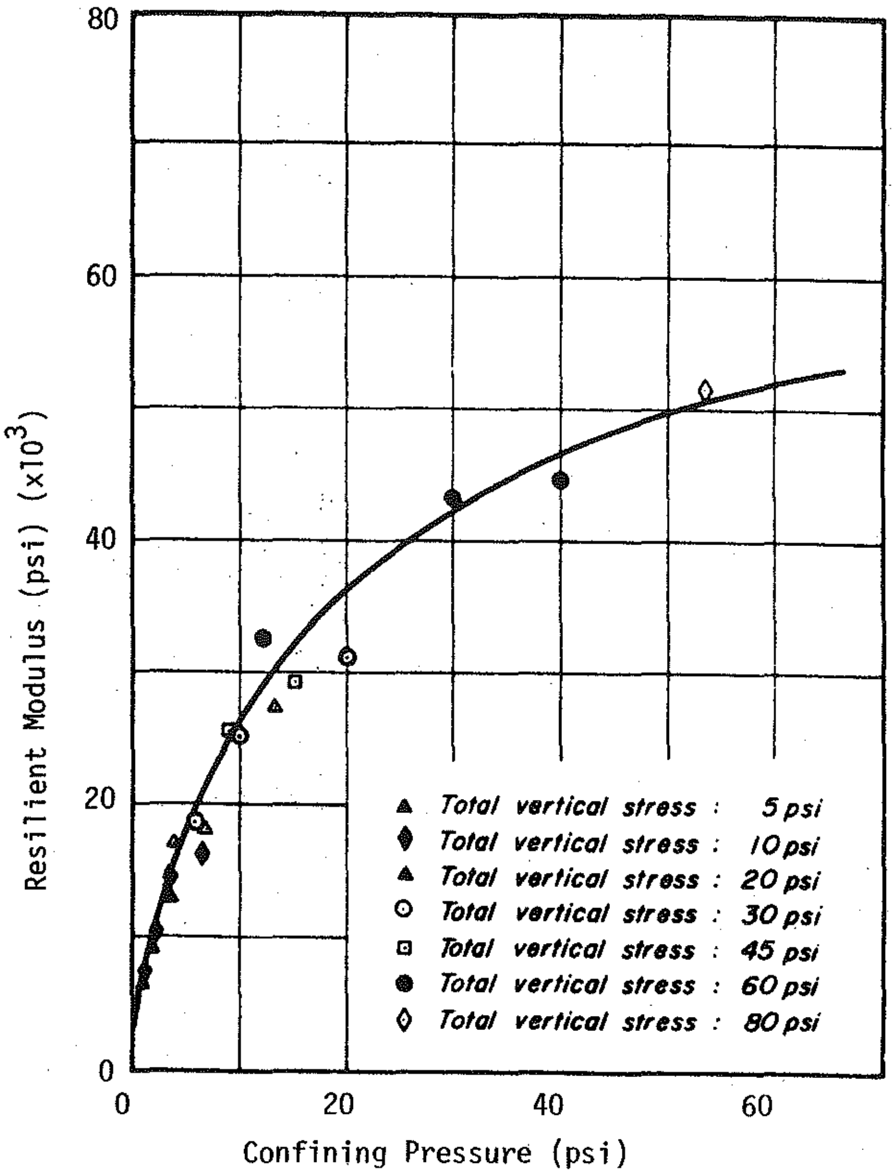


FIGURE 4.17 ARITHMETIC PLOT OF THE RELATIONSHIP BETWEEN RESILIENT MODULUS AND CONFINING PRESSURE FOR GRAVEL,(43).

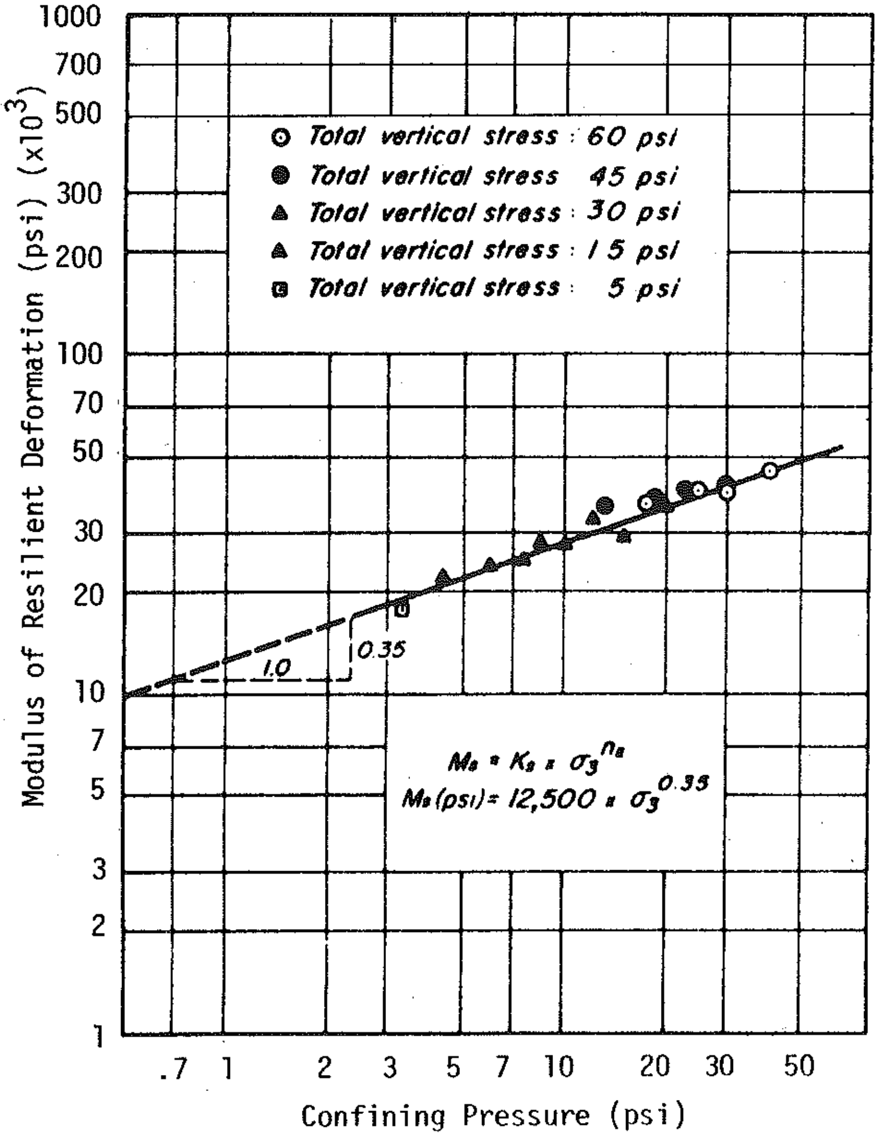


FIGURE 4.18 LOG-LOG PLOT OF THE RELATIONSHIP BETWEEN RESILIENT MODULUS AND CONFINING PRESSURE FOR SAND, (43).

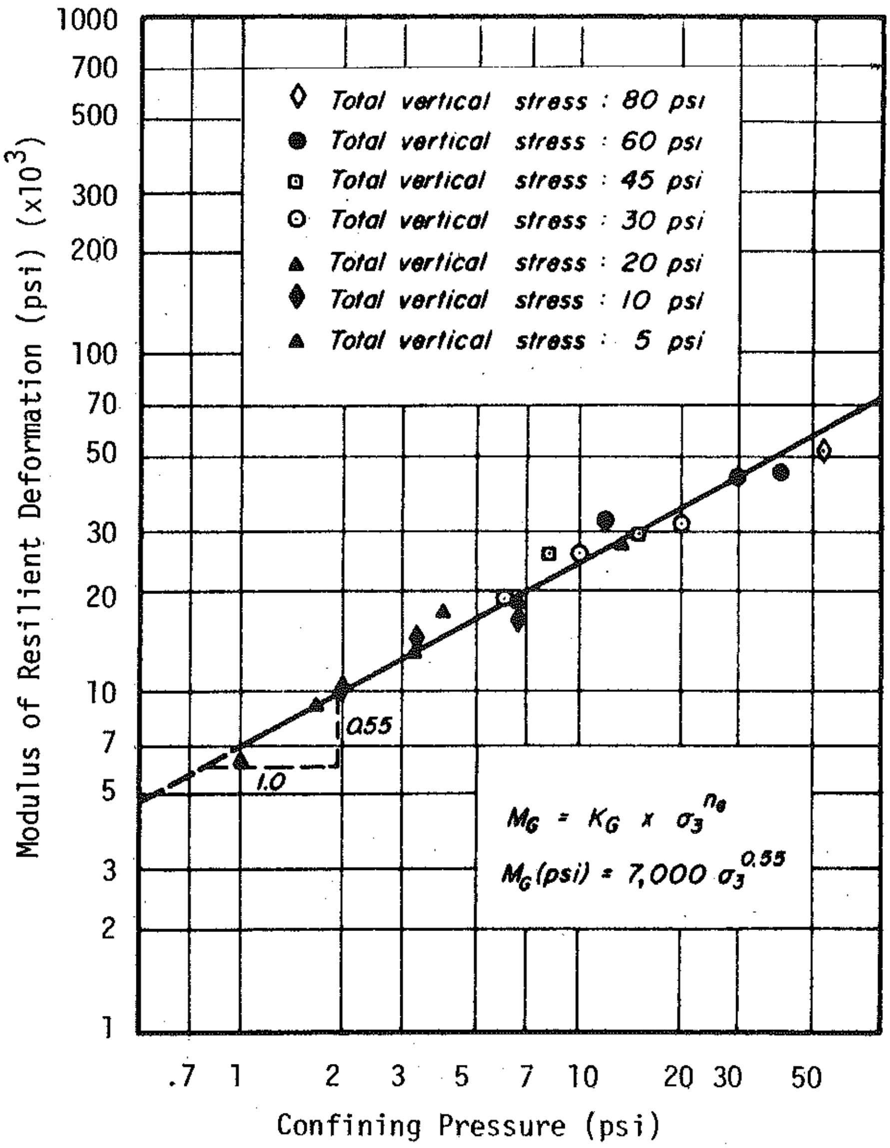


FIGURE 4.19 LOG-LOG PLOT OF THE RELATIONSHIP BETWEEN RESILIENT MODULUS AND CONFINING PRESSURE FOR GRAVEL, (43).

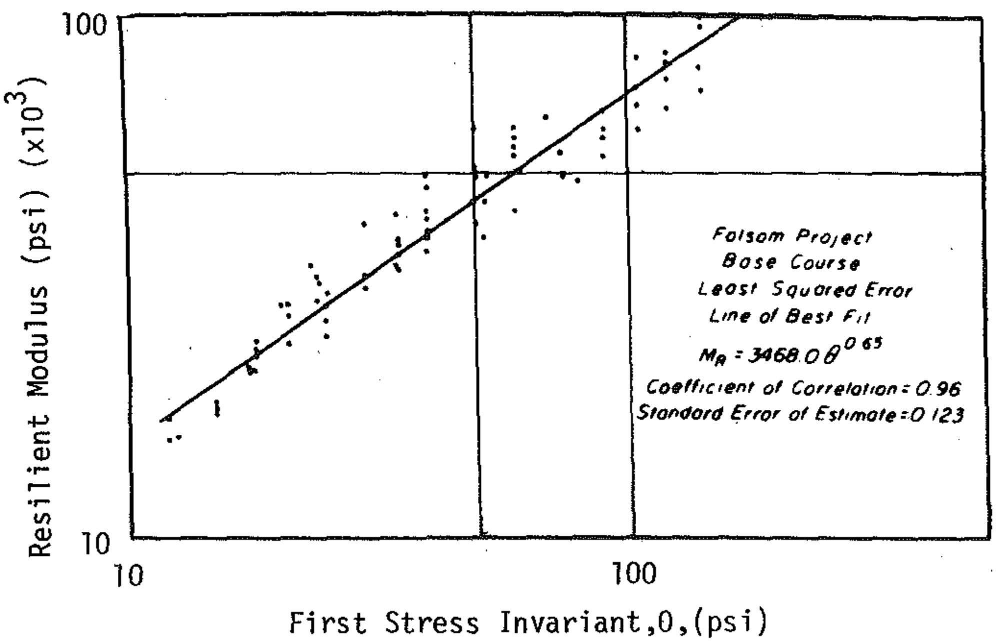
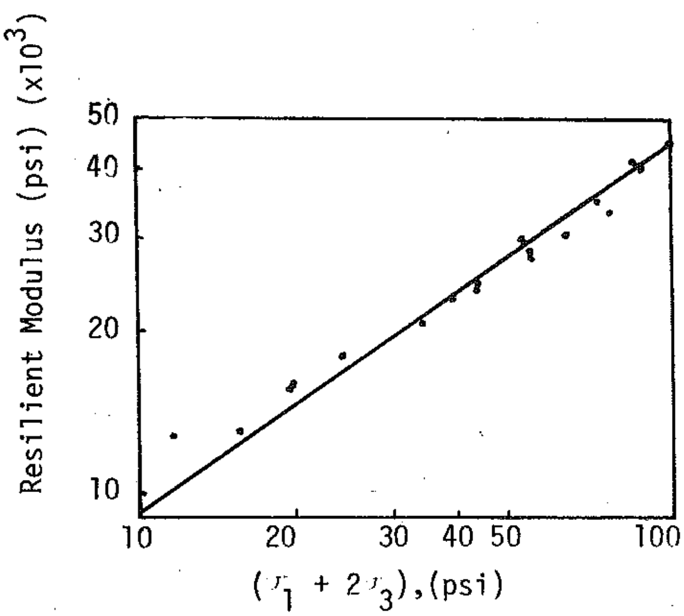


FIGURE 4.20 RELATIONSHIP BETWEEN RESILIENT MODULUS AND SUM OF PRINCIPAL STRESSES, (38).

found that at a constant deviator stress, the axial strain decreased with increasing confining pressure. However, Poisson's ratio did not appear to be related to confining pressure. In contrast to Morgan's results, Hicks and Monismith (27) found that Poisson's ratio increased as the confining pressure decreased. In all cases, they reported the non-linearity of the stress-strain relationship increased as the confining pressure decreased.

Allen and Thompson (22) have compared results based on triaxial testing with a constant confining pressure (CCP), to those with variable confining pressure (VCP). They found that the resilient modulus values for the CCP tests were slightly higher than those for the VCP. Also, the permanent deformation in CCP tests was always greater than that in the VCP. Barksdale (23) found that the plastic strain, permanent deformation, decreased with increasing confining pressure. Tests associated with a current research project, sponsored by the Michigan Department of State Highways and Transportation, conducted by Baladi at Michigan State University, tend to confirm both results. Results obtained from a repeated load triaxial testing with a constant low confining pressure tend to be on the conservative side when they are used to study rutting in the flexible pavement which is associated with the accumulation of plastic strain in the surface and base course layers.

The least squares equation relating M_R to θ was found to be more accurate than that relating M_R to confining pressure. Analysis of test data revealed higher correlation coefficients and a lower standard error for equation (4.3) than for equation (4.2). The explanation for this is believed to be that equation (4.3) accounts for all 3 principal stresses, whereas equation (4.2) accounts for only 2 principal stresses. The constant confining pressure tests tended to overestimate the value of Poisson's ratio, which was believed to be attributable to the anisotropic behavior of the material and the increased volume change of the sample associated with the CCP test. Figures (4.21) and (4.22) show these results for the VCP and CCP tests, respectively. Elastic, isotropic materials cannot have a Poisson's ratio that exceeds 0.5, but this is always the case for the CCP test. Allen and Thompson concluded that the use of a constant value of Poisson's ratio of 0.35 to 0.4 is an adequate representation of this property for pavement deflection analyses.



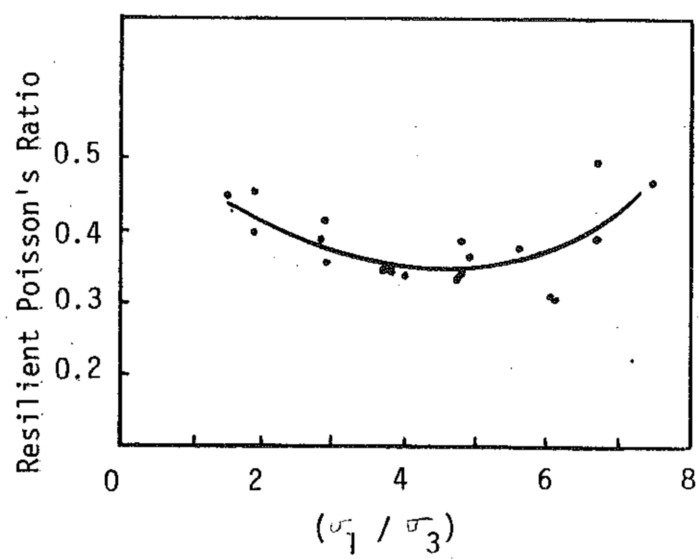
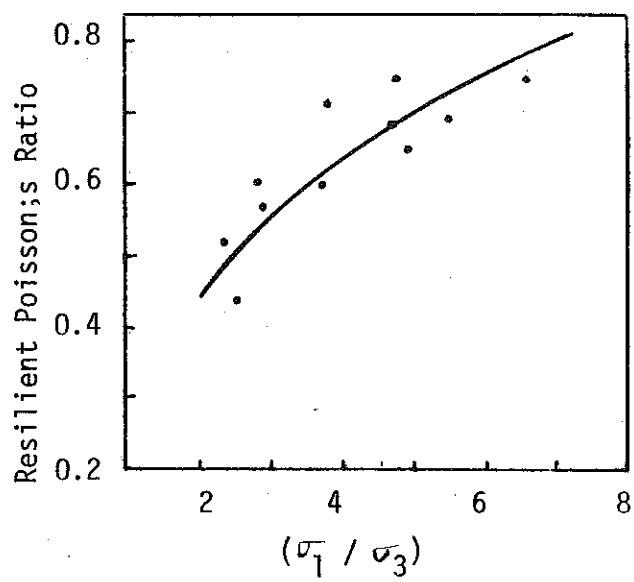


FIGURE 4.21 VARIATION OF RESILIENT MODULUS AND RESILIENT POISSON'S RATIO WITH THE SUM AND THE RATIO OF PRINCIPAL STRESSES RESPECTIVELY,(22).



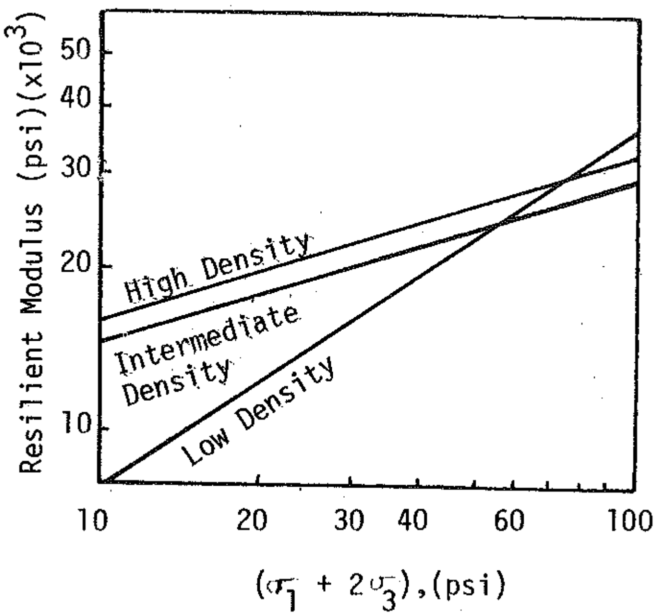


FIGURE 4.22 VARIATION OF RESILIENT POISSON'S RATIO AND RESILIENT MODULUS WITH THE RATIO AND THE SUM OF PRINCIPAL STRESSES RESPECTIVELY, (22).

2.5. Duration of Stress Application

Most investigators have concluded that the effect of the duration of the stress application on resilient response is negligible. Although the resilient modulus tends to increase as the time of load duration decreases, this effect is considered insignificant for the range of load durations encountered in pavement structures.

Hicks and Monismith (27) cite the work of Seed and Chan (6), on studies of the effect of load duration on the sample response for a silty sand. They found that for a decrease in duration from 20 minutes to 1/3 second, the resilient modulus increased from 23000 to 27000 psi, representing a change of 18 percent. They also showed that total deformation of the sample increased, for increases in duration up to 2 minutes. Hicks and Monismith (27) also confirmed the insignificance of load duration of 0.1 to 0.25 seconds.

Barksdale and Hicks (23) found that the resilient response of materials tested was only minimally affected by a variation of load duration from 0.04 to 1.0 second. They concluded that the sample response is independent of the duration of stress application, and that any stress pulse duration in the range of those applied to the pavement by moving wheel loads may be used in the lab with reasonable accuracy.

2.6. Rate of Deformation

It has been determined that resilient modulus tends to increase as the rate of deformation of the sample increases. Trollope (59) found that the resilient modulus increased 20 percent, as the rate of deformation increased from 0.002 to 0.040 inches per minute. Seed et al (43) reported similar findings. Researchers have concluded that effect of this parameter on the resilient response is insignificant, since the change in resilient modulus was negligible for such a large range of variation in the rate of deformation.

2.7. Frequency of Load Application

The effects of varying frequency of load applications also appears to be insignificant. Although the resilient modulus has been found to both increase and decrease with changes in frequency, the magnitude of

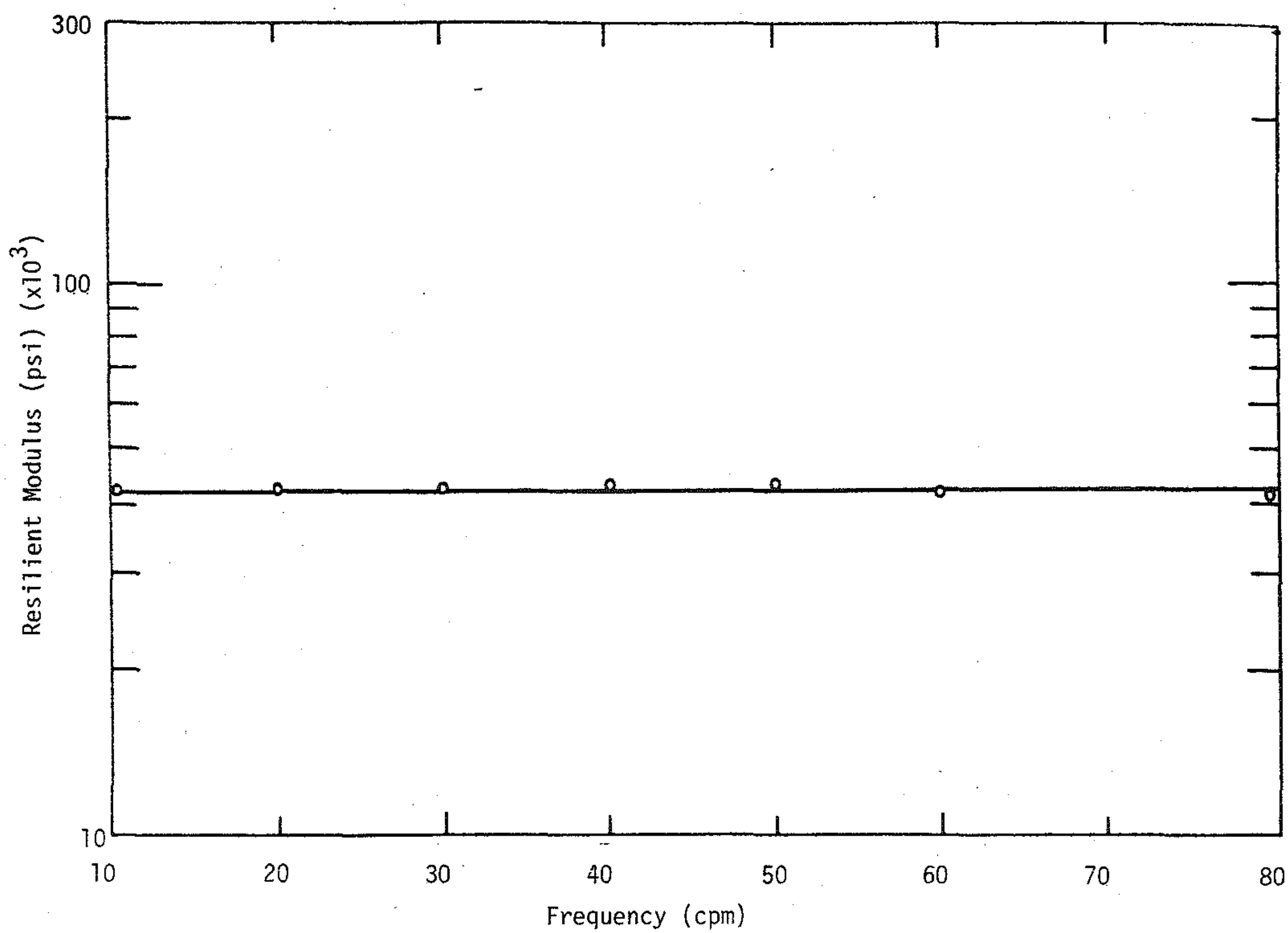


FIGURE 4.23 EFFECT OF LOAD FREQUENCY ON RESILIENT MODULUS FOR σ_3 = 5 psi AND σ_d = 15 psi, (31).

the change is small. In tests on silty sands, Coffman (62) found that the resilient modulus increased with frequency. The increase was on the order of 50 to 100 percent, depending upon the water content and density of the sample. Tanimoto and Nishi (46) reported conflicting results: the increase of resilient axial strain (decrease of the resilient modulus) with increasing frequency of loading. At higher numbers of stress repetitions, approximately 30,000, the effect of frequency was not discernable.

Kalcheff and Hicks (31) found that frequency changes had no effect on the resilient modulus, for tests on coarse aggregate as illustrated in Figure (4.23). They stated that any reasonable frequency of loading may be used to determine the resilient characteristics of cohesionless soils.

2.8. Type of Aggregate and Gradation

It appears that the effects of aggregate type and gradation are fairly insignificant in comparison to the effects of stress state. However, results of testing in this area are inconclusive and not well defined. Haynes and Yoder (26) conducted tests on gravel and crushed stone and found that for a given degree of saturation, the crushed stone samples exhibited greater total and resilient deformations than did the gravel. For both aggregate types, increasing the percent of fines passing the #200 sieve had no effect on the resilient modulus.

The relationship between the resilient modulus and the percent of fines is unclear as reported by Hicks and Monismith (27) and Barksdale and Hicks (23). For a range of confining pressure from 0 to 10 psi, they reported that the resilient modulus for partially crushed aggregate decreases, and that of crushed aggregate increases, as the percent of fines is increased. The changes in the resilient modulus in these cases were slight. Hicks (29) also reported a slight decrease in resilient modulus with increasing fines content for the same test data. In particular, for equation (4.2), Hicks reported that K decreased for partially crushed aggregate and increased for crushed aggregate as the fines content of each was increased. In general, K for the crushed aggregate was greater than K for the partially crushed aggregate, at corresponding relative densities. This too is not clearly defined, but n was found to decrease slightly as percent of fines increased. Poisson's ratio decreased in most cases as fines content increased, and it was generally greater for the partially crushed aggregate. All of these trends are shown on Table (4.3).

Barksdale and Hicks (23) reported a significant increase in Rut Index and, hence, a tendency to rut as the percent of fines is increased. They suggested to minimize rutting in the surface course and to improve drainage in the base course, that as little fines as practical be used in the base course.

Allen and Thompson (22) also found that aggregate type and gradation had minor effects on the resilient response. Although resilient modulus values were slightly higher for crushed stone samples as compared to gravel, Poisson's ratio was found to vary minimally between materials, and they concluded that the effect of material type and gradation were far surpassed in importance by the effects of stress level.

2.9. Void Ratio

Good agreement has been reached with respect to the effect of this parameter upon the resilient response. The resilient modulus increases as the void ratio decreases (dry density increases) Poisson's ratio is affected slightly, but shows no consistent variation with changes in void ratio.

Trollope (59) noted a 50 percent increase in resilient modulus from loose to dense sand samples. Mitry (36) and Hicks (29) found that Poisson's ratio decreased slightly as the void ratio decreased. However, the effects of density on the resilient modulus were reduced by increasing the fines content as shown in Figure (4.24).

More recently, Barksdale and Hicks (23) found that rutting of the surface increases 1 1/2 to 2 times as a result of a decrease in density from 100 to 95 percent of ASSHO density during construction. Allen and Thompson (22) reconfirmed the established relations between resilience and density in their tests with constant and variable confining pressures. Resilient modulus increased with decreasing void ratio, and Poisson's ratio was not significantly affected.

TABLE 4.3 INFLUENCE OF AGGREGATE GRADATION ON RESILIENT PROPERTIES OF GRANULAR BASE MATERIALS, (27).

Passing # 200 %	Relative Density,%	Degree of Saturation %	$M_R = K_1 3^{K_2}$		Mean Poisson's Ratio	Materials
			K_1	K_2		
3	89.2	0	11,752	.53	0.45	Partially
5	85.5	0	10,252	.64	0.45	
8	86.5	0	8,939	.61	0.34	Crushed
3	79.9	100	9,598	.55	0.25	
5	83.8	100	9,430	.50	0.35	
8	81.5	100	9,063	.52	0.25	Aggregate
3	89.3	0	12,338	.55	0.41	Crushed
5	87.0	0	13,435	.56	0.27	
10	86.0	0	14,672	.50	0.27	
5	77.2	0	11,446	.59	0.35	
10	77.0	0	14,313	.52	0.23	Aggregate

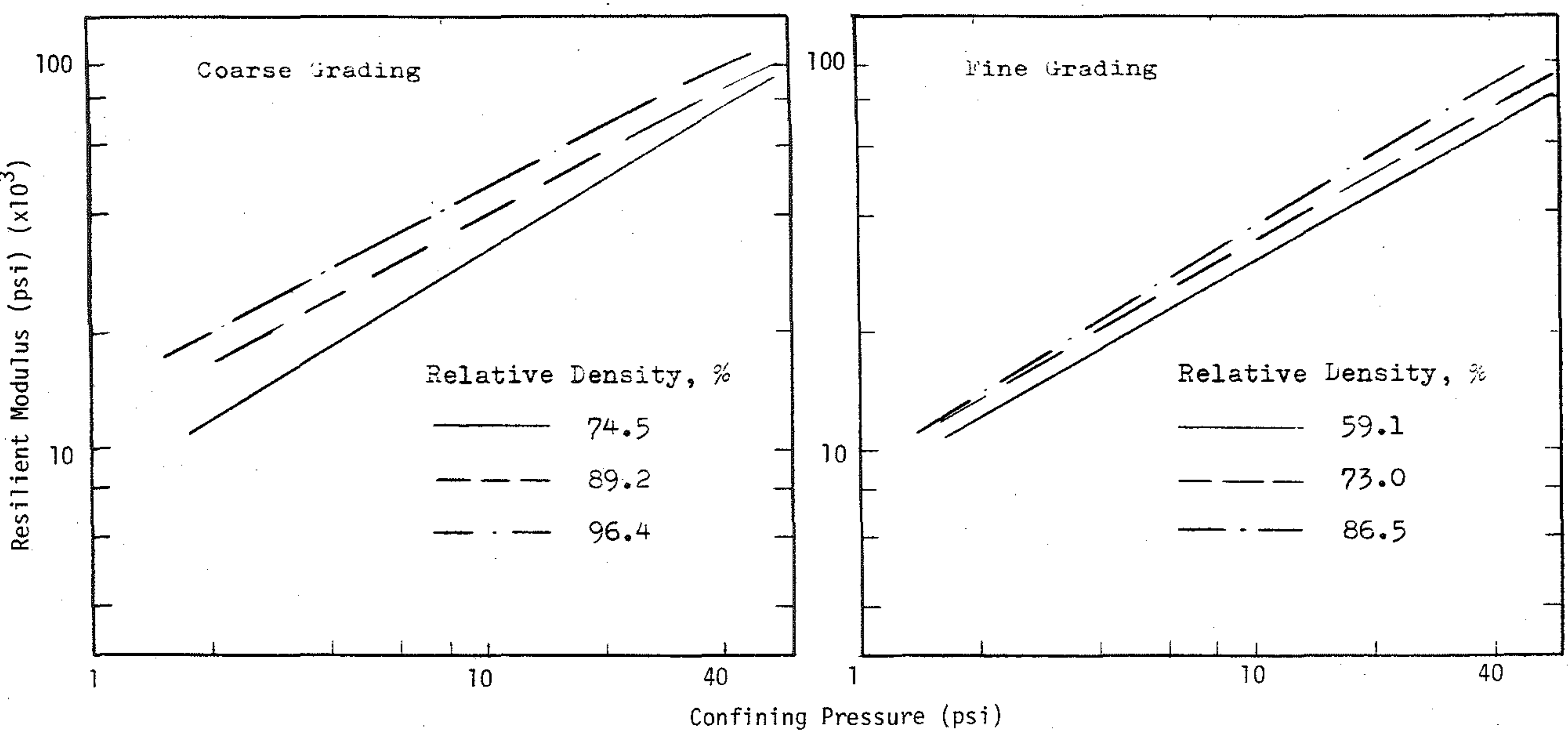


FIGURE 4.24 EFFECT OF DENSITY ON RELATIONSHIP BETWEEN RESILIENT MODULUS AND CONFINING PRESSURE, (27).

2.10. Degree of Saturation

The nature of the effect of the degree of saturation upon the resilient response of test samples is complex, and the extent of its effect appears to be related to many other parameters such as aggregate type and test drainage conditions. In general, as the degree of saturation increases, so does the resilience, and the resilient modulus is reduced. This was determined conclusively by Haynes and Yoder (26). For gravel specimens, they reported that as the degree of saturation increased from 70 to 100 percent, the resilient modulus decreased by 50 percent. For crushed stone, a change of saturation from 70 to 80 percent caused a 20 percent decrease in resilient modulus.

Hicks (29) found that as the degree of saturation increases, Poisson's ratio decreases. It was noted in test results that Poisson's ratio was always less than 0.5. Theoretically, Poisson's ratio should equal 0.5 for undrained test conditions where there is no volume change. Since it was not, Hicks concluded that this was due to improperly functioning LVDT's or sample inhomogeneity. He concluded that based on a total stress analysis, as saturation increased, Poisson's ratio decreased, and the resilient modulus was only slightly affected.

Comparisons of results at a given confining pressure based on effective stresses indicated that the resilient moduli for saturated specimens tested under undrained conditions were approximately the same as those determined for dry specimens. Saturated samples tested under drained conditions had slightly higher resilient moduli than those under undrained conditions. This is believed to be the result of pore water pressure build-up in undrained tests. Partially saturated samples had the lowest moduli of all as shown in Figure (4.25).

It was expected that the resilient modulus would continue to decrease as saturation increased, but this was not shown by the results. Hicks explained that the reason for this inconsistency is related to the manner in which the results were compared. The dry and partially saturated data were compared on the basis of total stresses whereas the dry and saturated data were compared using effective stresses. He stated that if all results were compared in terms of total stresses, the resilient modulus would steadily decrease as the degree of saturation

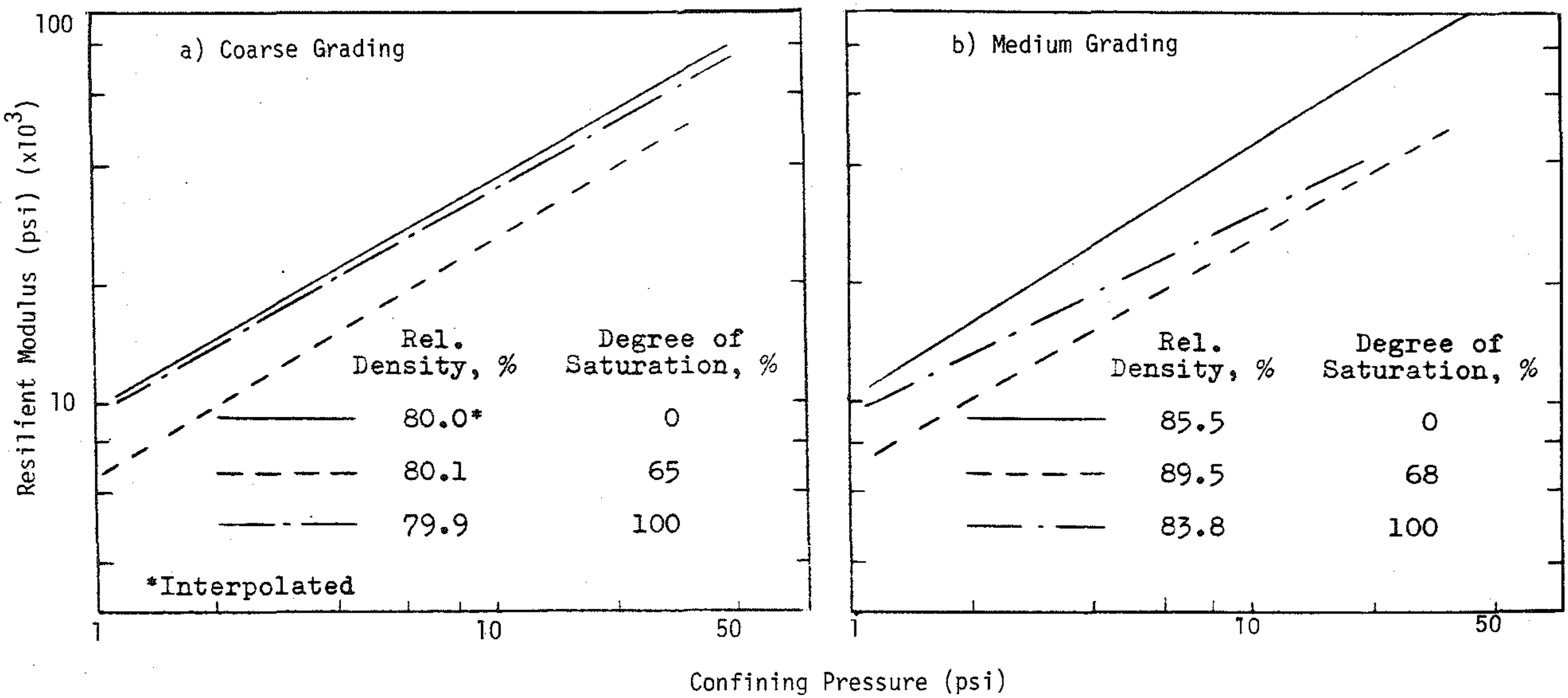


FIGURE 4.25 EFFECT OF DEGREE OF SATURATION ON THE RELATIONSHIP BETWEEN RESILIENT MODULUS AND CONFINING PRESSURE, (27).